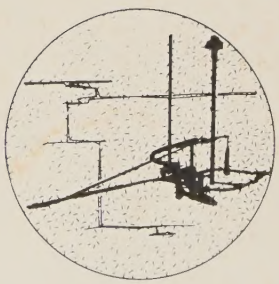
STRIAE

HYDRAULIC PROPERTIES AND GROUNDWATER BALANCE IN A SOIL-ROCK AQUIFER SYSTEM IN THE JUKTAN AREA, NORTHERN SWEDEN

TOMMY OLSSON

1980



VOLUME 12

UPPSALA 1979

Issued by ------- SOCIETAS UPSALIENSIS PRO GEOLOGIA QUATERNARI

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STRIAE, a Monograph Series for Quaternary Studies

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STRIAE 12

Hydraulic Properties and Groundwater Balance in a Soil-Rock Aquifer System in the Juktan Area, Northern Sweden

By
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Doctoral Dissertation
to be Publicly Discussed in the Lecture Hall of the Institutes of Geography and Geology on January 18, 1980, at 10 a.m., for the Degree of Doctor of Philosophy (according to the Royal Proclamation No. 327, 1969).

SOCIETAS UPSALIENSIS PRO GEOLOGIA QUATERNARIA

UPPSALA 1979

ISBN 91–7388–015–9
ISSN 0345–0074

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Hydraulic Properties and Groundwater Balance in a Soil-Rock Aquifer System in the Juktan Area, Northern Sweden

TOMMY OLSSON

STRIAE

Olsson, T., 1979 12 15: Hydraulic Properties and Groundwater Balance in a Soil-Rock Aquifer System in the Juktan Area, Northern Sweden. *Striae*, Vol. 12, 72 pp. Uppsala. ISBN 91–7388–015–9, ISSN 0345–0074.

Large groundwater inflows and high hydrostatic pressures in the rock mass during the construction of the Juktan hydro-power plant, made it necessary to determine the hydraulic properties of the main aquifers in the area. The hydraulic conductivity of the rock and the soil cover as well as the recharge to these formations have been determined. The obtained parameter-values are combined into a water balance study. In this connection, the groundwater balance under both natural and disturbed conditions have been studied. It was found that the drainage of the rock due to the tunnelling caused an increase in recharge to the rock aquifer by a factor of 3 when the rock mass was drained. Determinations of the distance of influence of the drainage show that the groundwater level was lowered at considerable distances away from the tunnel. By means of water head measurements, the hydraulic anisotropy of the rock mass was obtained. These measurements show that the Revsund granite is anisotropic with a ratio of 1:3 between the main directions. It has been possible to connect both this anisotropy and the to depth decreasing hydraulic conducitivity to the present state of stress in the rock mass.

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Introduction

The investigations were made in the construction area of the Juktan power plant in northern Sweden. They form part of a hydrogeological research-program continuously carried out by the Swedish State Power Board. The main part of the program consists of studies of the hydrogeological processes and groundwater phenomena, paying special attention to the technical and practical effects on the construction. At the Juktan power plant the hydrogeological investigations were promoted by the inflow of large quantities of water into the underground excavations and by technical problems arising from high hydrostatic pressures in the rock mass.

Our knowledge of the nature of groundwater flow and groundwater occurrence in the till soils and crystalline basement rocks is limited, despite the enormous extent of these formations in Sweden. However, in recent years the interest in the hydraulic properties of the crystalline rocks increased as a result of expansion of subterranean constructions, but also as a result of the efforts made for radioactive waste storage in rock.

Theoretical studies of fractured rock hydrology provided insight into the significant hydraulic properties. The concept of parallel plate flow is widely applied to the analysis of fracture flow in both individual fractures and multi-fractured rock masses. Works on this field have been done by Lomize (1951), Snow (1965) and Louis (1967),

who produced equations for the fracture flow based on both theoretical and empirical relations. The scale of the problem is of importance for the modeling of the fractured rock mass, i.e. when continuous or discontinuous approaches must be used. Rats and Chernyashov (1965) studied the scale of the problem and related it to the degree of heterogeneity of the fractured rock.

For many years it was assumed that fractured rock generally functioned as a homogeneous isotropic porous medium. It later became evident however, that the fractures caused a directional conductivity in the rock which gave rise to hydraulic anisotropy. Warren and Root (1963) discussed this property of fractured oil reservoirs and its effect on the nature of flow, and presented an idealized model for characterization of a fractured medium.

It has long been known that the hydraulic properties of rock masses are different at different depths. Generally the superficial rock mass is far more conductive than at greater depths. Davies and Turk (1964) and Snow (1968b) demonstrated that this condition is dependent on a decrease in fracture frequency and apertures with increasing depth. This effect was recently related to the mechanical properties and stress conditions of the rock. This relation was discussed by Brace *et al.* (1968), Pratt *et al.* (1977), and others.

Groundwater investigations of crystalline basement rocks in Sweden dealt mainly with individual hydraulic parameters. Basic knowledge of rock hydraulics was contributed by Larsson (1968 and 1972) as regards the groundwater conditions in relation to brittle fracturing and fracture genesis.

G. Bergman (1972) studied the groundwater recharge and L. Carlsson and A. Carlstedt (1977) evaluated the hydraulic conductivity in different rock types. A. Carlsson and T. Olsson (1976 and 1977a) presented studies of hydraulic conductivity and its relation to depth and defined the hydrogeological properties of fractures (A. Carlsson and T. Olsson, 1980b). Several authors deal with water inflow into tunnels and underground constructions, attributing the inflow to either the hydraulic conductivity of the rock mass (M. Bergman 1977, Lindblom 1979 and others) or to various technical problems arising from the flooding (Morfeldt *et al.* 1967, Sund *et al.* 1977 and others). Only a few studies have been made of the groundwater balance in hard rock aquifers, and of the interaction between the soil and rock aquifers (Hörnsten *et al.* 1977, Wedel 1978).

Williams and Farvolden (1967) and Knutsson (1971) proved that discontinuities, such as fissility and sediment lenses, are of importance for the water-bearing capacity of till soils. However few works deal with the hydraulic properties of till soils. Basic information was presented by Ekström (1934), Knutsson (1971) and Engqvist *et al.* (1978). The infiltration capacity of different soil types, including tills, was studied by von Brömssen (1968).

Determinations of specific hydraulic features of a geological formation may not be representative of the formation as a whole. The degree of representativity depends chiefly on the method used but also on the hydraulic properties of the formation. The water-pressure tests can represent this state of affairs. In a rock mass with low conductivity the value obtained for the hydraulic conductivity in fact represents only a small volume of the rock outside the drill-hole, while measurements of a rock mass with high conductivity give values which represent a comparatively large rock volume (L. Carlsson *et al.* 1979a). The penetration in low conductivity formations and thereby the representativity of the result is improved if the hydraulic pressure is increased or the measuring time prolonged. However, increased pressure may expand existing fractures (A. Carlsson and T. Olsson 1979a) or propagate new ones (Haimson and Fairhurst 1970), while prolonged measuring times are inconvenient. The same disadvantages also apply to other methods of determining hydraulic conducitivity, and the results obtained represent only the scale in which the determination was made.

To get an over-all check of the results, their validity must be verified by regarding the values not merely as values but also as parameters governing the actual groundwater conditions.

The main aim of this work is to define the hydrology of the main water-bearing strata, till and crystalline basement in the Juktan area, and combine these hydraulic properties into a water balance study. The reason for this treatment is to study the representativity in different scales of the properties obtained with various methods. The water balance study also gives an opportunity of determining the interaction between these main aquifers. In addition, as both rock mechanical and hydrogeological studies were performed, the relation between the hydraulic properties and the stress situation of a fractured rock mass will be discussed. The study as a whole concentrated on the hydraulic properties of the rock mass, while the till soil was cursorily examined.

As the investigations were associated with the construction of a subterranean hydro-power plant it was also possible to study the effects of the construction on the groundwater system and vice versa.

General description of the Juktan area

Investigation area

The Juktan power plant is situated in the discharge area of River Umeälv, about 20 km northwest of Storuman municipality in the county of Västerbotten. It is a subterranean pumped storage power plant, with an extensive tunnel-system, linking it to three separate reservoirs. Figure 1 illustrates the tunnel-system and the investigated area.

The present investigations were performed in the tunnels and rock chambers of the plant as well as on the ground surface. The tunnel-system constitutes the natural limits of the investigated area. In the north and the south the area is defined by the Lakes Storjuktan and Storuman, and along the tunnel-system the area is connected with the extension of the various tunnels. This produces an area about 10 km wide by about 20 km long.

The tunnel-system penetrates an elongated area at high altitude, running NW–SE, delimited in the south and north by the valleys of the Umeälven and Juktån. Thus there are major differences in altitude along the tunnels, as illustrated in Fig. 2. Lake Storuman represents the lowest level in the area, being about 350 m above sea level, while Mt Storblaiken in the centre is the loftiest summit at an altitude of about 750 m.

The Juktan area is situated on Topographical Map 23 H Stensele. The geological maps of Västerbotten County in-

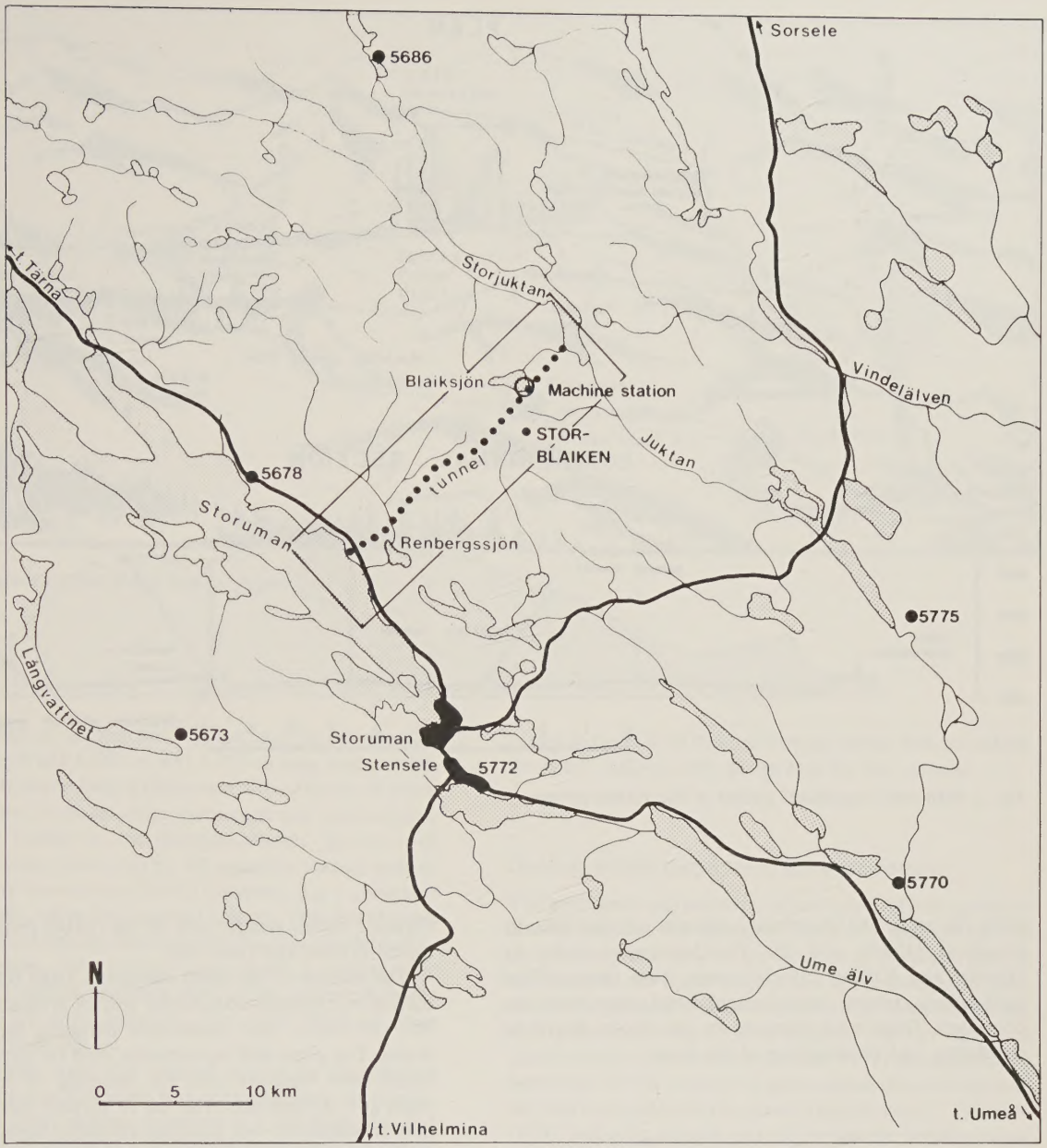


Fig. 1. The situation of the tunnel-system of the Juktan power plant and the investigated area.

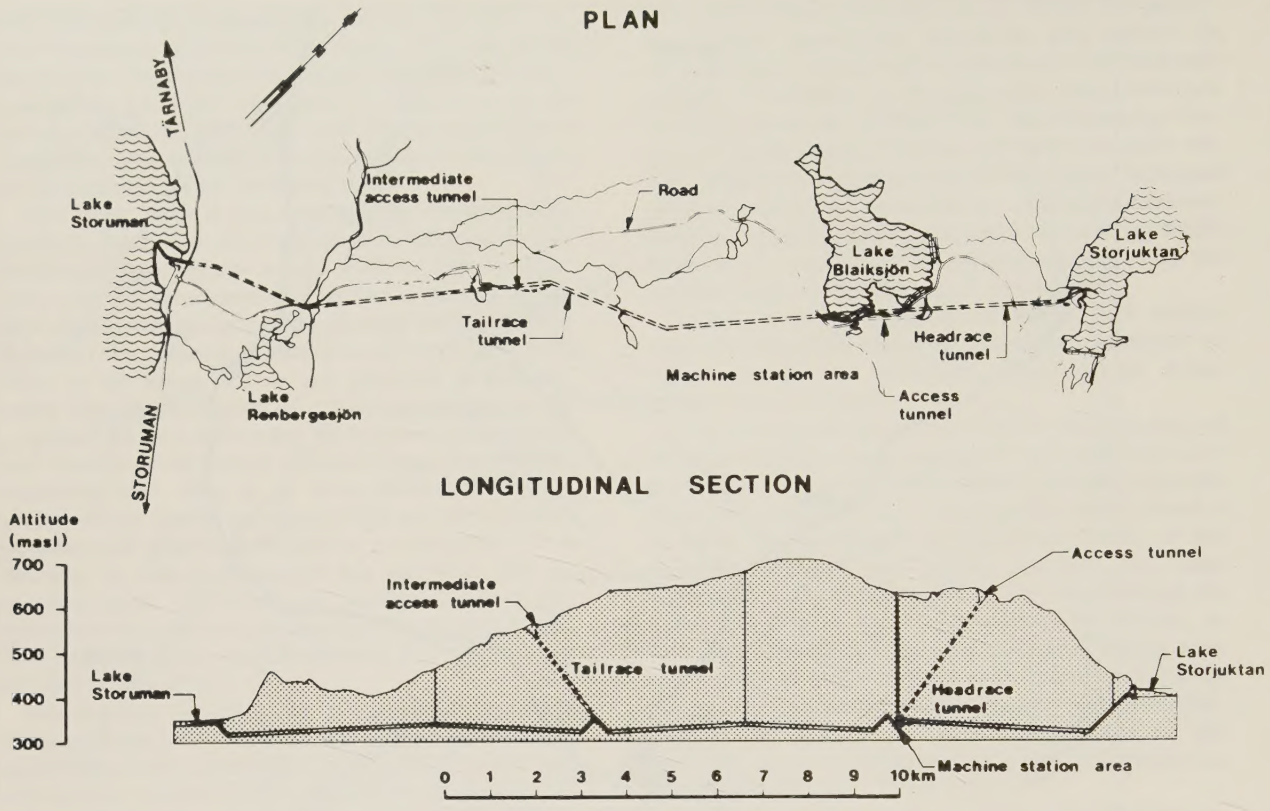


Fig. 2. Plan and longitudinal section of the Juktan project.

clude the area. The Quaternary deposits are described by Granlund (1943), and the Pre-Quaternary rocks by Gavelin and Kulling (1955). Apart from these official publications, several special unpublished maps were also consulted. These were prepared by the Power Board for the design and construction of the plant.

The power plant

The pumped storage scheme. The Juktan power plant is a pumped storage power station, which draws on three separate reservoirs. Water from the regulated Lake Storjuktan is pumped through the machine station for storage in an upper reservoir in Lake Blaiksjön. During the production of electric power, the water stored in Lake Blaiksjön is discharged through the machine station to an outlet in Lake Storuman. A reversible pump-turbine connected to a generator is used for pumping of water and generation of electricity.

The plant can also be used without transfer of water to Lake Storuman, in that during generation the water is returned to Lake Storjuktan, i.e. like a conventional pumped storage plant with two reservoirs. Furthermore, a by-pass tunnel allows transfer of water from Storjuktan to Storuman without pumping and generation. In this case, and during normal operation, the unexploited water discharged into R. Juktån can be used by power plants along R. Umeälv, between Lake Storuman and the tributary. Figure 3 shows general view of the Juktan project with its tunnel-system and reservoirs.

The volume of the upper storage in Lake Blaiksjön (40 million m^3) permits continuous energy production during 80–120 hours, and continuous pumping for 100–115 hours. The plant will be normally used for production 12 hours each weekday, and for pumping 10 hours every night and at weekends. The use of surplus energy at night and at weekends, will promote an useful improvement of the electric energy. Due to losses during pumping and generation, the plant produces no net energy, notwithstanding that the net head is higher during production than during pumping. Nevertheless the transfer of water and its use in the existing hydro-power plants along R. Umeälv produces 140 GWh/year net.

Construction works. Construction began in 1973 and the plant was commissioned in late 1978. Due to the design of the plant, the main part of the construction works consists of underground excavations including tunnels of considerable length. The rock excavated amounts in all to about 2 million m^3.

The headrace tunnel from Lake Storjuktan to the machine station is about 5,000 m long having a cross-sectional area of 60 m^2. The length of the tailrace tunnel from the machine station to the outlet in Lake Storuman is about 14,650 m and the cross-sectional area 80 m^2. The

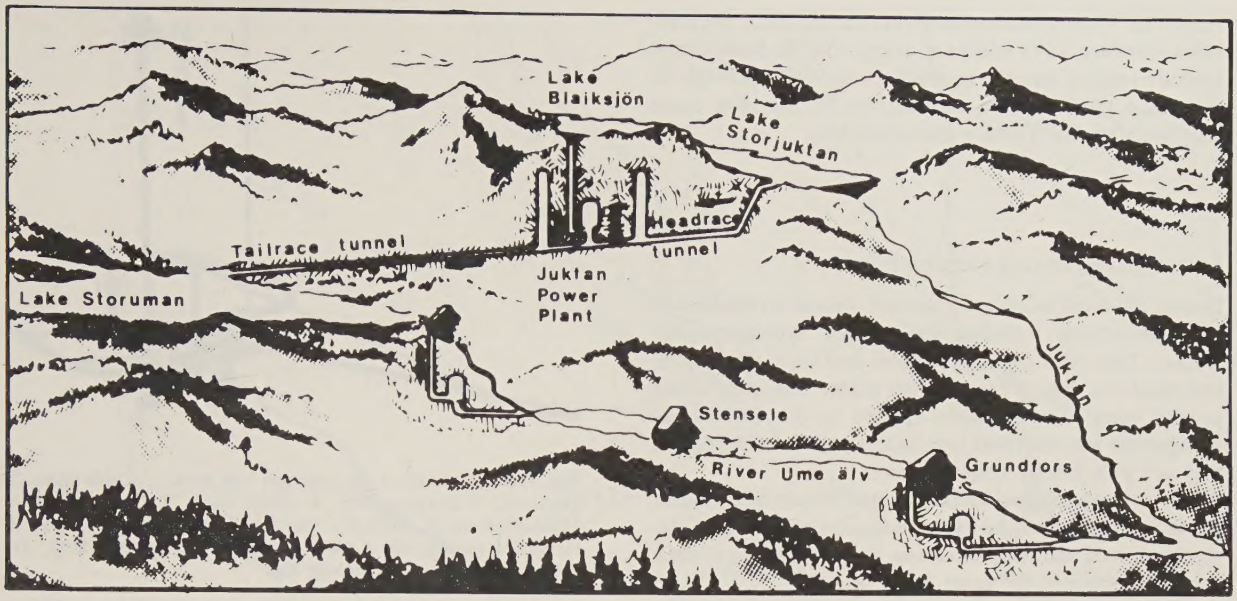


Fig. 3. A general view of the Juktan project.

access tunnel at the machine station and the intermediate access tunnel are 1,780 m and 1,325 m long respectively. The tunnels of the Juktan plant have a total length of about 25 km, which includes those mentioned and some smaller tunnels in vicinity of the machine station. Several rock chambers were excavated in the machine station area to accomodate diverse mechanical functions, e.g. a generator hall, butterfly valve hall, control hall etc. The complicated rock constructions are illustrated by the scale model in Fig. 4. Furthermore, three vertical surge shafts, a vertical cable shaft and a vertical penstock were drilled using a Robbins full facer (2,4 m in diameter). The total length of these shafts is about 750 m.

Excavations were also carried out in the Quaternary deposits. Most of this work was connected with the regulation of the upper storage reservoir in Lake Blaiksjön, but also with the building of a temporary road along the tunnel system.

Three rock filled dams were constructed to convert the natural Lake Blaiksjön in an upper reservoir. The dams consist of an impervious core of till and shoulder-fill of rock. Altogether, 0.5 million m^3 till were used for this purpose, the till being quarried from neighbouring pits. The reservoir has a volume of 40 million m^3 and an amplitude of 11 m.

Site investigations

The site exploration for the power plant began in 1970 and was completed when the construction started in 1973. The exploration was executed by the staff of the Power Board, and various contractors. During construction, engineering geological studies and special investigations were carried out. The present hydrogeological investigations were initiated in the mid 1975 when tectonic zones with considerable water inflows were penetrated by the tunnels.

Geological and geophysical site explorations

The geological and geophysical site explorations consist of diverse projects, the results of some of which are used in the present work. Due to the location of the plant, deep in the bedrock, the major part of the site investigation compriced geophysical measurements. An airborne radiomagnetic survey covering an area of about 200 km^2 along the tunnel-system (Werner and Granar 1972) determined the final route of the tunnels. In some places the airborne survey was complemented by ground geophysics (L. Eriksson 1974), including seismic and radiomagnetic investigations. In the present work, the geophysical results are used to chart the tectonic features in the area.

During this exploration, detailed geological studies were made of some areas of the bedrock and soil cover. The aim of the rock survey was to locate and assess the quality of a purative boundary between two rock types (graphitic schist and granite) in the north of the area (A. Carlsson 1972a), while the soil cover was examined to evaluate local till resources as construction material for the regulation dams, and gravel for use in concrete (A. Carlsson 1972b).

The rock mass was investigated by drillings (Moberg 1973 and pers. com.). One 318 m-deep hole was drilled (core-drilling) in order to examine the rock mass in which the machine station was situated and a further 23 to a depth of less than 70 m were made in the machine station area and at the dam sites. Beside these main drillings a great number of shallow investigation holes were made at the initial points on Lakes Storjuktan and Storuman, and at the two access tunnels. No drillings were made along the tunnel-system outside these areas. The collected rock-cores were logged with regard to rock type and existing

fractures. Water-pressure tests were carried out in the drill-holes (Moberg 1973 and pers. com.). On the basis of the fracture logging and the hydraulic tests it was possible to determine the hydraulic conductivity of the rock mass together with the fracture characteristics.

Investigations during construction

During the construction of the plant diverse investigations were performed, including the present hydrogeological studies. They also cover geological and rock mechanical investigations, which are to some extent also used in the present work. The methods applied in the various investigations are described later in this paper and, therefore they are only briefly reviewed here.

The hydrogeological investigations include field studies primarily to discover the hydrogeological properties of the main aquifers, and determinations and evaluations of data primarily obtained for other purposes.

Basic information on the geological and hydrogeological conditions is derived from existing maps and descriptions (topographical and geological maps), from the documentation of the exploration, and from technical reports. The main geological and hydrogeological features of the area were thus established, and only minor supplementary field studies were carried out.

Information on the groundwater levels in the area and the water-bearing capacity of the main aquifers were obtained from the investigation drillings and from water wells drilled in rock. Loggings of the groundwater level allowed estimation of the extent of groundwater drainage due to the tunnelling.

The groundwater leakage into the tunnels was expelled by pumping at four points in the tunnel-system. At two of these points the water flow was continuously measured, by means of a V-notched weir connected to a recorder. These measuring stations are situated at the access tunnel to the machine station, and at the intermediate access tunnel. The water leakage in the headrace tunnel and in the southernmost stretch of the tailrace tunnel was pumped out through the openings of these tunnels. The water flow at these points was measured at intervals. In addition to these measurements of the total leakage, measurements were taken of local water-bearing zones in the tunnel-system.

The hydraulic properties of the aquifers were determined by various hydraulic tests. Hydraulic conductivity of the till soils was assessed by laboratory tests using permeameters, while the conducitivity of the rock mass was measured *in situ*. For this reason two main methods were applied, firstly water-pressure tests in drill-holes during exploration and secondly, determinations based on measurements of the water leakage in the tunnels.

The static water head above the various excavations in the machine station area were continuously measured at drill-holes in the roofs of the excavations. Altogether 38 drillholes 10 m deep were used for this purpose. The measuring points were also used for hydraulic tests of the interconnection between the existing fractures and for inflow measurements.

In situ rock stress measurements were carried out at three points in the machine station area (Hiltscher 1975), and two in the headrace and tailrace tunnels. The tritium content and radiocarbon dates of water samples from the tunnels were analysed in order to evaluate the transformation velocity of the groundwater.

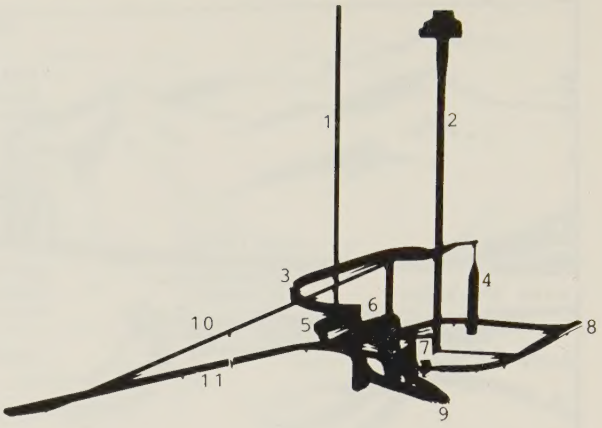


Fig. 4. Scale model of the machine hall area. 1. Cable shaft, 2. Penstock, 3. Surge gallery, 4. Surge shaft, 5. Control hall, 6. Machine hall, 7. Butterfly valve hall, 8. Tailrace tunnel, 9. Headrace tunnel, 10. Access tunnel to the surge gallery, 11. Access tunnel to the machine hall.

The present investigation to a great extent comprises calculations and determinations based on the obtained results. Various modes of calculation were employed, including analytical and numerical methods using the finite element principle.

Geology

The crystalline basement

The sketch map in Fig. 5 illustrates the distribution of different rock types forming the Pre-Quaternary basement in the area. Thus the tunnel-system traverses bedrock consisting mainly of Revsund granite. In addition, metamorphic rocks of sedimentary origin, chiefly foliated greywackes and schists, occur in the northernmost stretch of the headrace tunnel and east of the tunnel-system. The Revsund granite incorporates remnants of the sedimentary sequence. Pegmatites and diorite are also found, mainly as layers and dikes intersecting the granite. Some km east of the tunnel-system, one of the Swedish Caledonian nappes, the Blaik Nappe, rests on a basement of Revsund granite.

The rocks along the tunnel-system belong to the Svecokarelian orogenic zone. The oldest rocks are the metamorphosed sedimentary rocks in the northern tunnel-system. According to Welin (1970:444) the age of this sequence is unknown, but has been estimated as *c.* 1900–2100 Ma. This sequence consists of supracrustals, mainly of metagreywacke type. The schistose rocks included in this sequence are graphite bearing and foliated with a direction of about NW–SE dipping 30°–60°N (A. Carlsson 1972a:2). According to T. Lundqvist (1973:5) the Revsund granite is considered to be transitionally ser- to postorogenic. Recent radiometric datings (Welin 1970:444) yield an age of 1785 Ma.

The Revsund Granite is a massive rock of rather undifferentiated and monotonous type. There is no noticeable foliation and the characteristic planar strucures are fractures.

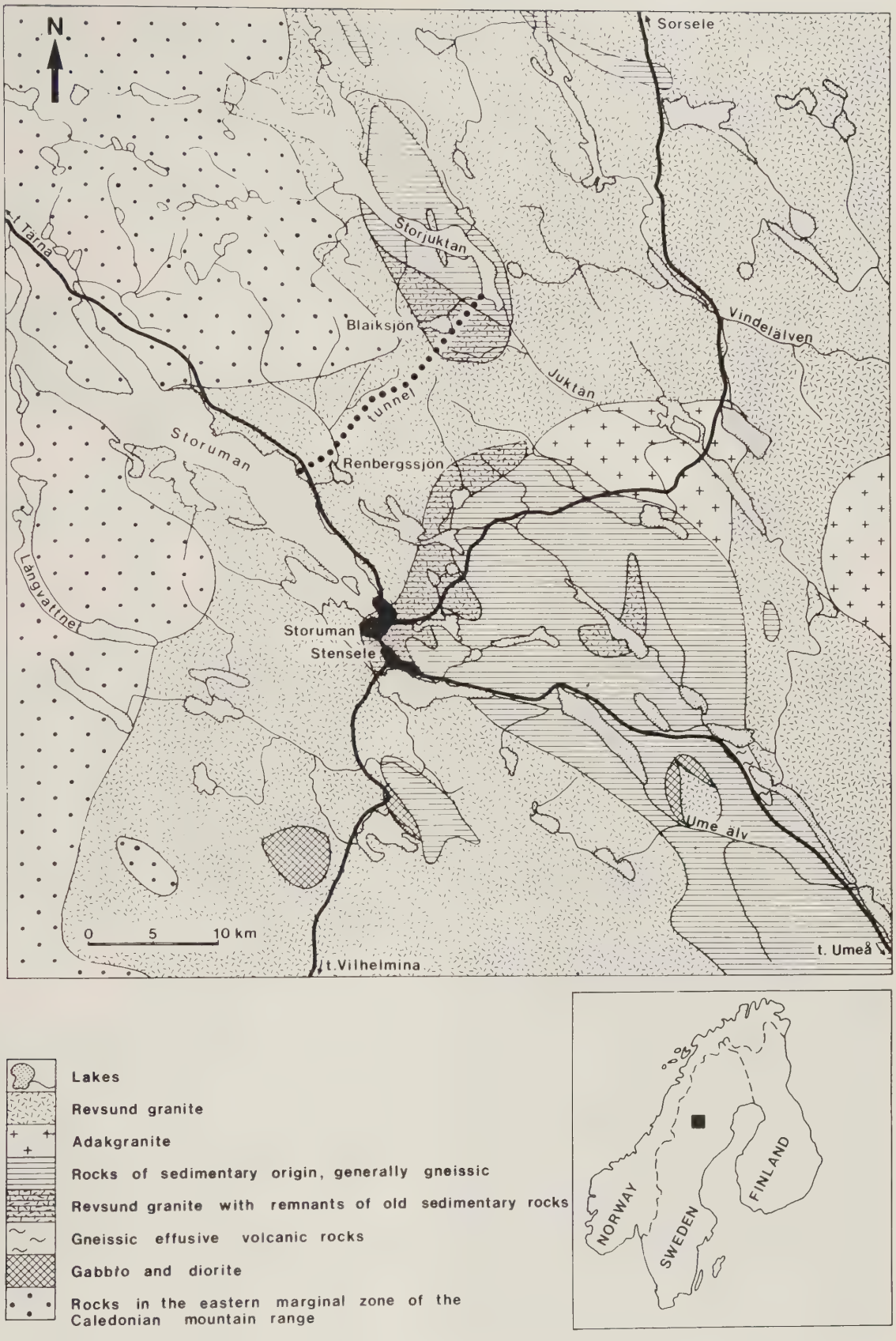


Fig. 5. Sketch map of the Pre-Quaternary geology of the Juktan area. Modified after Gavelin and Kulling (1955).

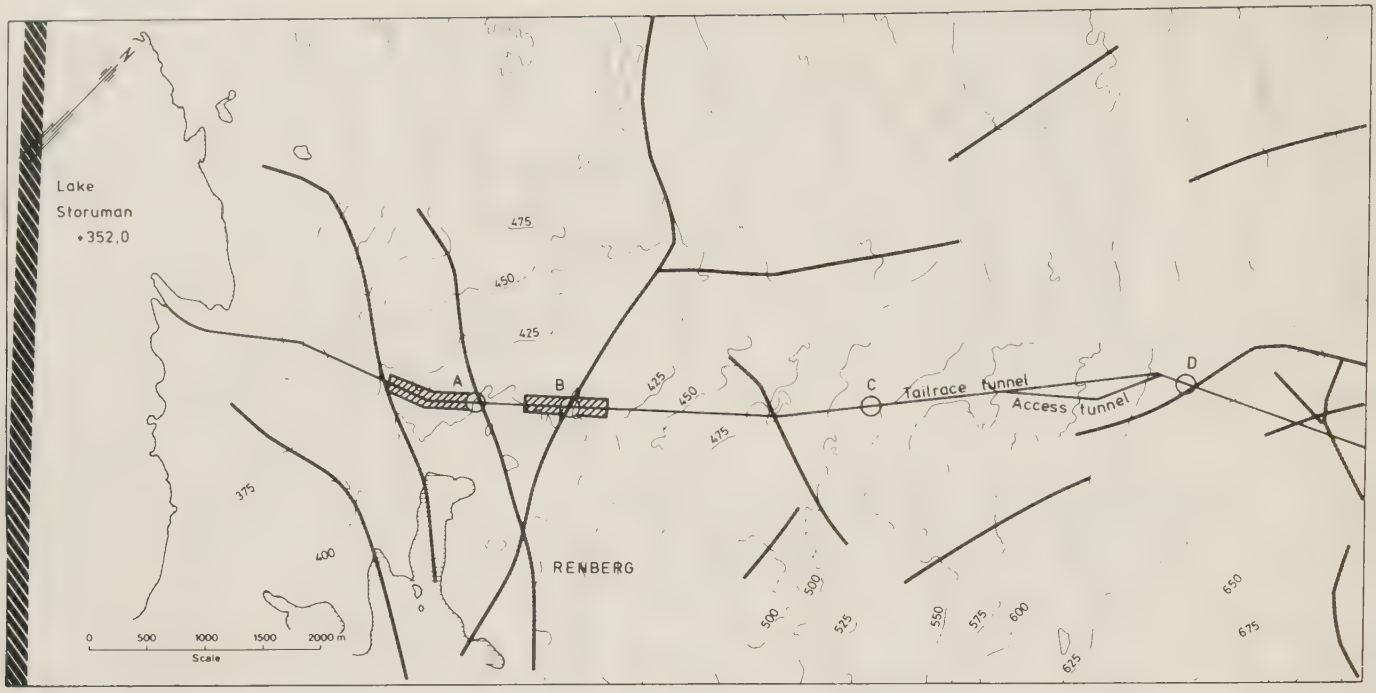


Fig. 6. Main tectonic structures of the area along the tunnel-system. Areas in the tunnels with large volumes of water inflow are marked

Tectonic features. The main tectonic features along the tunnels are shown in the sketch map in Fig. 6. This map is based partly on geological observations during the hydrogeological investigations and partly on geophysical measurements, such as airborne radiomagnetic measurements and seismic investigations. Two main fracture valleys running NW–SE, dominate the area, and the intervening rock massif in which the tunnels are excavated, contain tectonic zones also bearing NW–SE. Some minor faults also indicate zones striking E–W and NE–SW. Normally, the tectonic zones are steeply dipping although flat zones are also found, principally in the vicinity of the intact high altitude area of Mt. Storblaiken.

The fractures in the Revsund granite constitute a well-developed, orthogonal joint system, composed of four sets, one horizontal or subhorizontal and three vertical or subvertical. The vertical sets have the following main strike directions:

1. N–S–N20°E
2. N40°E–N60°E
3. N80°W–N80°E

Figure 7 illustrates the joint pattern in the Revsund granite.

The fracture frequency, and consequently the fracture spacing, is different in the superficial rock mass and its deeper lying parts as seen from Table 1. In the uppermost part the rock is far more heavily fractured, with a spacing of about 0.1 m as a mean from the core logging (Moberg 1973). The denser fracturing in the superficial rock mass is a common phenomenon, especially in granitic rocks (cf. Johnson 1970:357 and A. Carlsson 1979).

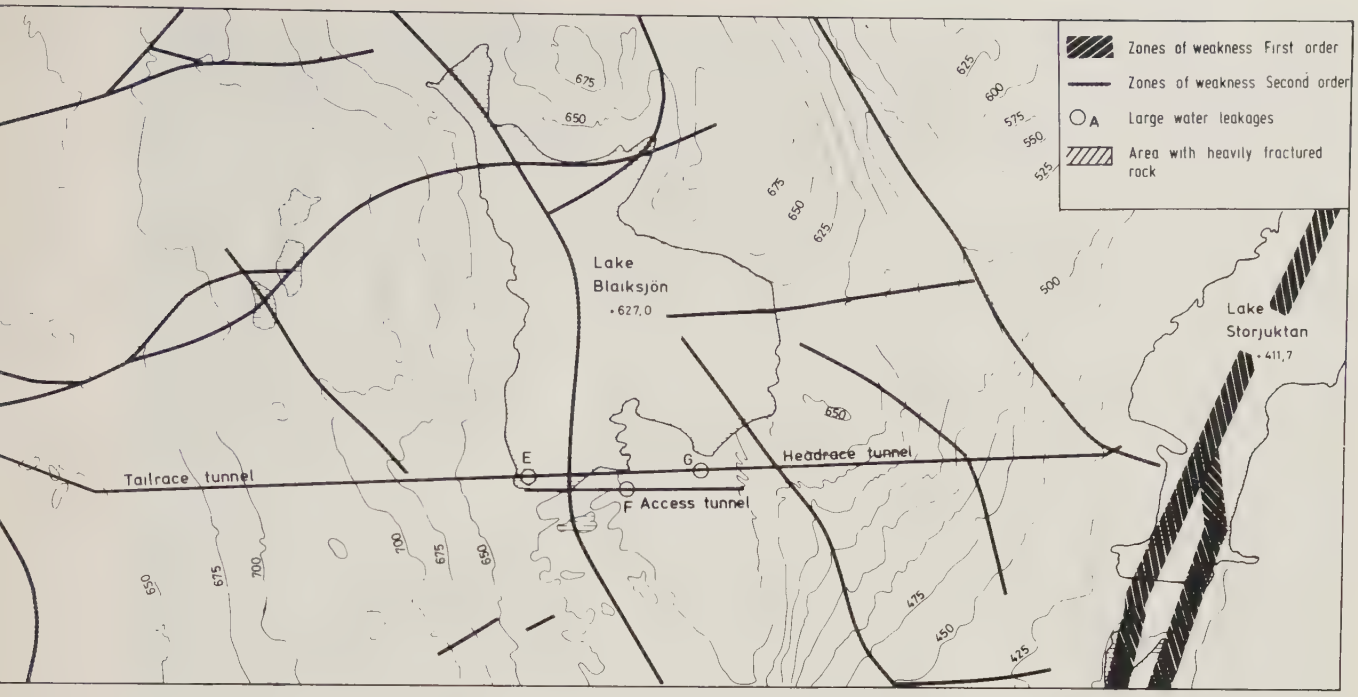
Quaternary deposits

The geological map of the Quaternary deposits in Västerbotten County (Granlund 1943) was used for the description of the soil types. Detailed investigations during site exploration were made by A. Carlsson (1972b), but covered only the northernmost part of the area, between the Lakes Blaiksjön and Storjuktan. In areas where supplementary information was needed, studies were carried out to provide information on the extent of different deposits and their interior structure. The distribution of the Quaternary deposits in the area is shown in Fig. 8.

Till is the predominant Quaternary deposit, although large areas are peat covered. Some 80–90 per cent of the area is covered with either till deposits or peat bogs. The residual 10–20 per cent is made up of exposed bedrock or glaciofluvial deposits. Glaciofluvial deposits of gravel and sand are found mainly in the south of the area, but are of limited extent.

Till deposits. The till soils in the area are mainly coarse grained and rich in boulders, but finer tills are present as well. The local basement strongly affected the petrographical and granulometric composition of the till. The till soil on the granite is in general coarser than in the area of metasediment. Moreover, the morphology of the till deposits is influenced by the basement. In areas of high altitude, the till deposits form a fairly thin, compact cover conforming with the rock surface, while in the lowlands it is loose, and less dependent on the basement morphology. In the vicinity of Lake Blaiksjön, the till deposits form transverse ridges of Rogen type, as seen in the aerial photograph in Fig. 9. The material in the ridges is compact and lenses of silt or sand are common. Figure 10 shows the granulometric composition of some till deposits.

The internial structure of the deposits varies from comparatively homogeneous to formations rich in lenses of sorted material. Flat fractures are also common, especially



A–G (A. Carlsson and T. Olsson 1978a).

in the more compact deposits. In the transverse ridges at Lake Blaiksjön, both lenses and fractures are frequent. Figure 11 shows a till cutting with a dominant horizontal structure due to fissility.

On the basis of seismic investigations and drillings, the thickness of the till deposits was determined as about 7 m on average. This value derives from 308 measurements distributed throughout the area. Yet the thickness varies considerably, the maximum being 20 m recorded in the basin of Lake Blaiksjön. The till cover is generally less thick at higher altitudes.

Glaciofluvial deposits. Gravelly and sandy glaciofluvial deposits are found to the north of Lake Renbergssjön in the south of the area. The deposits merge to coarser material as the distance from the lake increases. Additionally, there is an esker in the valley of River Juktån, just outside the actual investigation area. No sediments of silt or clay are known in the area.

Peat bogs. Wide areas are covered with peat, either as a thin cover resting on a basement of till or as thicker deposits filling depressions. In some larger basins the peat are of considerable extent and sedge marsches are common. The thin peat cover on the till dominates on the hilly terrain around Mt Storblaiken, indicating moist soil conditions.

Hydrometeorology and hydrology

Climatological records from seven measuring stations in the vicinity of the Juktan were used to define the hydrometeorological conditions in the investigated area. The climatological stations are operated by the Swedish Meteorological and Hydrological Institute, and included in the official meteorological network in Sweden. There are four stations less than 25 km away from the Juktan area, and a further three within a distance of 50 km. Table 2 lists the climatological stations which contributed data, their altitudes in m above sea level, and records kept while the location of each station are shown in Fig. 1. The Stensele station possesses record from the representative period 1931–1960, which were used for reference. Records were

Fig. 7. Revsund granite intersected by an orthogonal joint system consisting of one sub-horizontal and two vertical sets of fractures. Photograph by A. Carlsson.

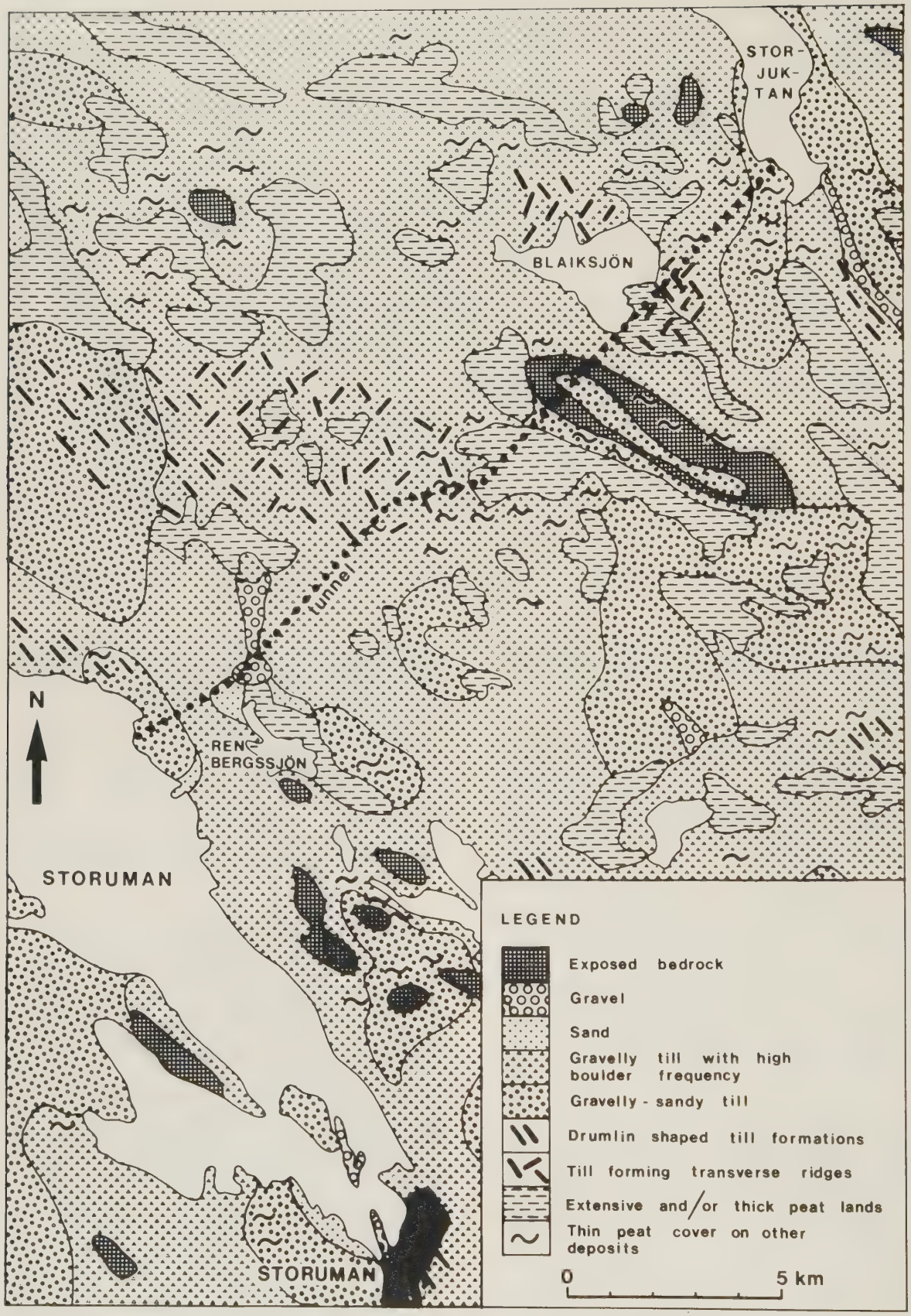


Fig. 8. Schematic map showing the main features of the Quaternary geology in the investigated area. Modified after Granlund (1943).

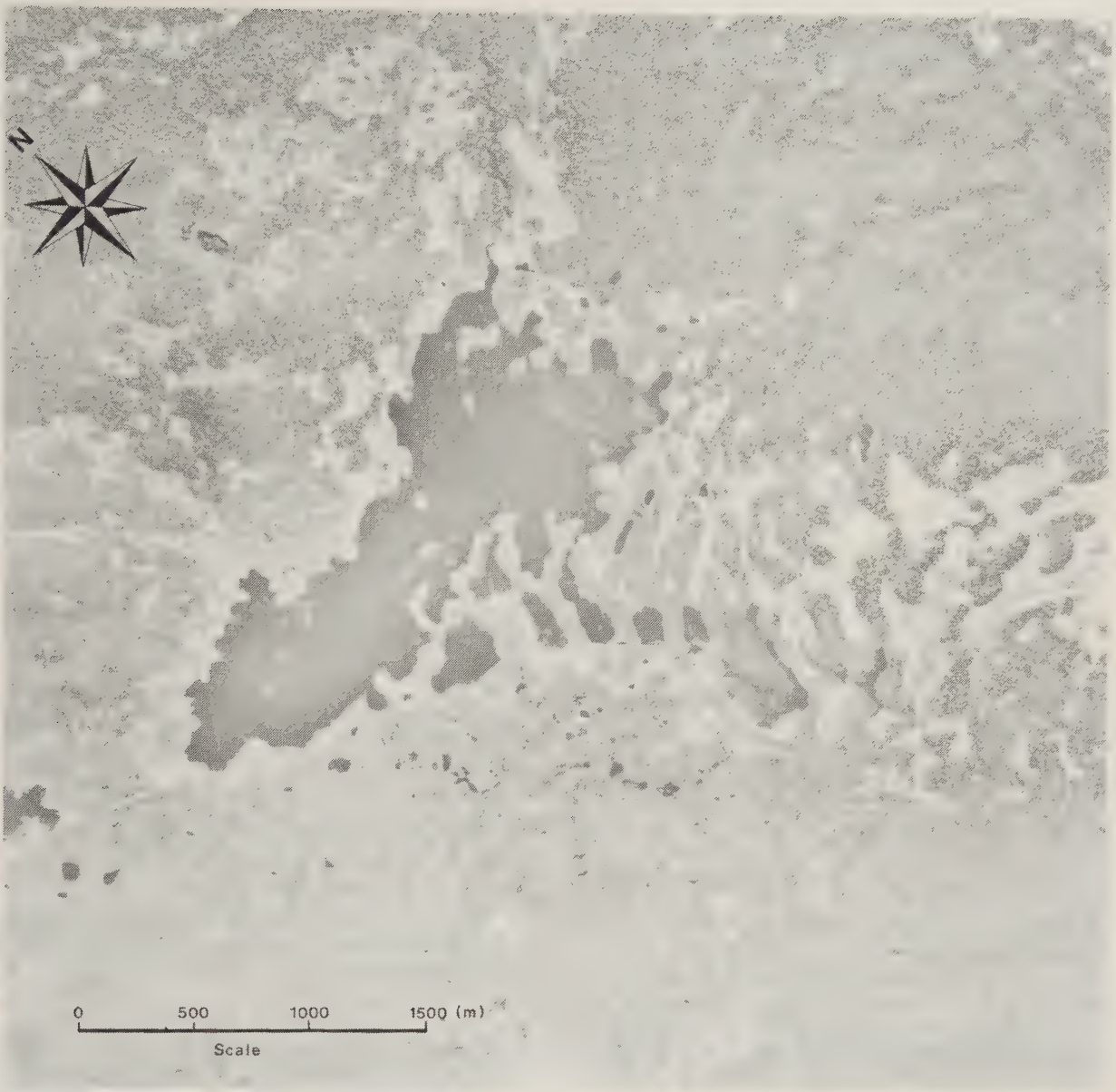


Fig. 9. Aerial photograph showing transverse ridges at Lake Blaiksjön. Photograph by National Land Survey Office 1972.

kept by all stations during the actual period of construction and investigation in the Juktan area. The calculations of the water balance are based on data from the period 1970—1978, i.e. from three years before the start of the construction until the plant came into operation.

The differences in altitude between the Juktan area and the climatological stations enforced conversion of the values of precipitation and temperature obtained at the stations. The river discharge was measured at three points in the main rivers, namely the outlet of Lake Storjuktan on R. Juktån, and the Umluspen and Gardikfors hydro-power plants on R. Umeälv. As these measuring points are associated with regulation storage in the rivers, the natural water discharge was calculated from the influx to the reservoirs. Data from the period 1970—1978 were used for the present study, and long term trends based on measurements in 1931—1960 served for reference.

Table 1. Frequency and spacing of horizontal fractures at different depths below the rock surface recorded from core-drillings in the area of Lake Blaiksjön (Moberg 1973).

Depth below rock surface (m)	Fracture frequency (l/m)	Fracture spacing (m)
0—5	9.0	0.11
5—10	8.6	0.12
10—15	13.2	0.08
15—20	9.8	0.10
20—25	3.6	0.28
25—30	3.0	0.33
30—35	1.2	0.80
35—40	4.2	0.24
40—45	10.2	0.10
45—50	0.8	1.25
50—55	0.6	1.67
55—60	2.0	0.50
60—65	3.2	0.31
65—70	0.2	5.00

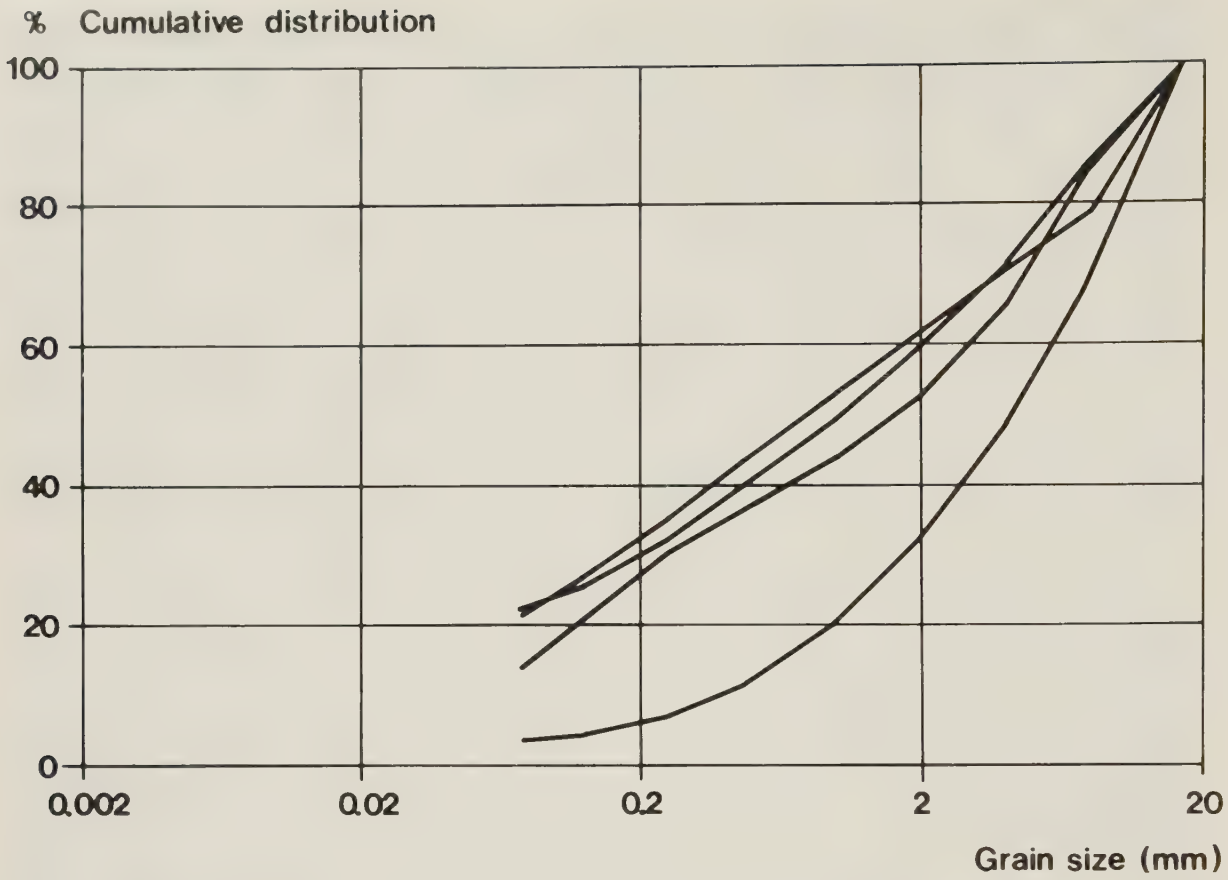


Fig. 10. Granulometric composition of material < 16 mm from some representative till deposits.

Precipitation

The precipitation was measured at all stations listed in Table 2. The differences in altitude have a major bearing on the precipitation recorded. The orographical effect was established by correlating the precipitation values from the different stations with their altitude. The mean precipitation during the period 1970–1978 is related to altitude in Fig. 12. A calculated mean value for the precipitation in the Juktan area is also included. Regression analyses of the mean annual precipitation values at each station and altitude reveal that the precipitation increases by about 50 mm/100 m, which gradient is comparable with results of other investigations. In his study of the water balance in the drainage area of Malmagen in River Ljusnan Melin (1943:13) observed a precipitation increase of about 25 mm/100 m. An increase in precipitation of 120 mm/100 m was recorded by Melin (1970:148) in the drainage area of Tåkern.

Monthly and annual mean precipitation figures obtained at the climatological stations during the period 1970–1978 are shown in Table 3. Calculated values for the Juktan area and long term values from Stensele are also included. The individual monthly precipitation values are presented later in connection with the water balance.

Table 3 shows the heaviest precipitation in July, but the figures are fairly high also in August to November. The monthly precipitation is least in winter, December to April. Compared with the long term records from Stensele, the actual period shows 3 per cent higher annual precipitation. The distribution through the year is similar in both the actual and the reference period, i.e. high summer and low winter precipitation.

Fig. 11. Till deposit at Lake Blaiksjön with a well defined horizontal structure due to fissility. Photograph by T. Olsson.

Table 2. Climatological stations in the vicinity of the Juktan area. Location, altitude and measured data.

Location Name	SMHI no	Altitude m.a.s.l.	Records Precipitation	Temperature	Snow cover
Blaiken	5678	350	●		
Giltjaur	5686	435	●		
Gunnarn	5770	278	●	●	●
Lång-vattnet	5673	420	●	●	
Sadiliden	5775	400	●		
Silverberg	5680	530	●		
Stensele	5772	326	●	●	●
Juktan area		555			

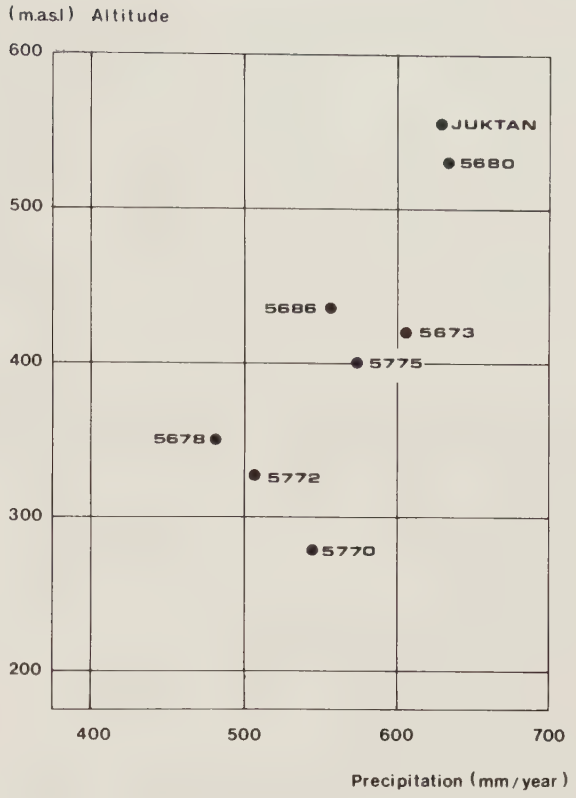


Fig. 12. Precipitation versus altitude for the seven climatological stations in the area. The legend of the numbers is given in Table 2.

In order to estimate the available precipitation it is of interest to know when the precipitation falls as snow. In winter the precipitation is stored in the snow cover and inavailable to the hydrological cycle until the spring thaw. The snow cover was measured at two stations, as seen in Table 2. Accordingly a continuous snow cover has a mean duration of 190 days during the actual period. The mean maximum thickness was 75 cm. In general, the snow cover forms in the mid October and melts in mid May. Snow comprises about 40 per cent of the total annual precipitation. In the Juktan area with its higher altitude and lower temperature both duration and maximum thickness are probably greater than these values. The effect of the altitude on the duration and thickness of the snow cover was reported by Melin (1943:20), who found that its water content increases greatly with increasing altitude (70 mm/100 m in the Malmagen area).

The long term values from Stensele shows similar figures for duration and maximum thickness, i.e. a duration of 194 days and a maximum thickness of 69 cm.

Temperature

The temperature was recorded at three climatological stations (cf. Table 2). Like the precipitation the temperature is related to the altitude, but this relation does not unambiguously increase or decrease, but varies in different seasons. During the summer the temperature decreases noticeably with increasing altitude. In autumn this effect gradually declined and in November to January the temperature proved to increase with increasing altitude. In spring the temperature once again decreased with increasing altitude. These relations between temperature and altitude emerged from regression analyses of the local temperature record.

The elevation effect was taken into account in the calculations of the temperature in the Juktan area. The mean monthly temperatures obtained in the period 1970–1978 are shown in Table 4. The values obtained in the period 1931–1960 are shown for reference. Compared with the long term trend, the actual period shows lower temperatures during the summer and higher in winter, although the annual mean temperatures are identical.

Table 3. Mean monthly and annual precipitation values (in mm) of the period 1970–1978 from the climatological stations, and calculated precipitation values for the Juktan area. The long term values from Stensele are also included.

Station		J	F	M	A	M	J	J	A	S	O	N	D	Year
5678	Blaiken	31	25	18	28	24	50	94	51	61	31	35	34	482
5686	Giltjaur	35	27	25	38	29	57	94	53	65	40	53	40	556
5770	Gunnarn	37	29	24	40	35	58	88	59	56	37	48	33	544
5673	Långvattnet	41	31	27	40	31	56	110	64	69	45	58	38	610
5775	Sadiliden	42	30	22	42	37	58	96	60	58	42	50	35	572
5680	Silverberg	43	32	32	43	32	59	99	65	75	50	65	43	638
5772	Stensele	35	28	19	33	29	55	98	47	55	34	42	32	507
	Juktan	44	33	29	43	36	50	111	66	72	45	58	41	628
	Stensele 1931–1960	30	23	21	25	33	57	80	67	47	37	38	36	494

Table 4. Mean monthly and annual temperatures (°C) obtained at the measuring stations at Stensele, Långvattnet and Gunnarn for the period 1970–1978. Calculated values for the Juktan area and long term values from Stensele.

Station	J	F	M	A	M	J	J	A	S	O	N	D	Year
Stensele	−10.8	−10.7	−5.4	−1.0	6.0	11.9	13.9	11.8	6.3	1.0	−6.2	−8.9	0.7
Långvattnet	−10.5	−11.7	−5.4	−1.5	5.5	11.3	13.2	11.2	5.9	0.7	−5.9	−8.6	0.5
Gunnarn	−11.1	−10.7	−5.4	−0.9	6.4	12.4	14.3	12.1	6.4	1.0	−6.2	−8.9	0.8
Juktan	−10.3	−11.6	−5.9	−1.9	4.6	10.1	12.1	10.4	5.4	0.5	−5.7	−8.3	−0.2
Stensele	−12.2	−11.0	−6.8	−0.2	5.9	11.0	14.3	12.2	7.1	1.0	−4.2	−8.3	0.7

Table 5. Comparison between values of the potential evapotranspiration obtained via Thornthwaite's and Pennman's formulae from climatological data from Stensele for the period 1931–1960. Actual evapotranspiration according to Tamm's formula for the same period.

Method	Potential evapotranspiration (mm)												
	J	F	M	A	M	J	J	A	S	O	N	D	Year
Penman	0	0	9	34	68	93	86	50	23	0	0	0	363
Thornthwaite	0	0	0	0	44	71	86	76	51	12	0	0	340
Tamm (actual evapotranspiration)													242

Evapotranspiration

The potential evapotranspiration in the Juktan area was calculated from the temperature values. The empirical relation presented by Thornthwaite (1945) was used for this purpose. The validity of this relation in the Juktan area was tested by comparing the calculations based on the long term temperature values from Stensele with those submitted by Wallén (1966:795). Wallén calculated the potential evapotranspiration from the global solar radiation with the help of Penman's formula (Penman 1948). Table 5 contains the results of the calculations according to Thornthwaite's formula, and to Wallén. Both methods give similar total evapotranspiration througout the year. Yet the monthly values are different. The actual evapotranspiration according to Tamm's linear relation for evapotranspiration versus temperature (Tamm 1959:9) is also shown in the table, but this method seems to give too low a value.

The Penman method probably produces the most accurate description of the real potential evapotranspiration, while Thornthwaite's method is more of a rough approximation. Högström (1964:2) found that the Thornthwaite's method underestimates the potential evapotranspiration in all autumn and winter months in high latitudes. As shown by e.g. Aslyng (1965:298), Penman's formula gives too high a value for the potential evapotranspiration because of exaggeration of the yield from water in spring and summer. Nevertheless, Aslyng and Wallén admit that the Penman method is the best available for calculation of potential evapotranspiration from climatological data. However, it calls for comprehensive climatological information, which is recorded only at Stensele, and is thus inconvenient for use in the Juktan area.

The potential evapotranspiration at Juktan was calculated by means of Thornthwaite's formula. The actual evapotranspiration, and thus, the available precipitation (actual precipitation minus the evapotranspiration), was determined with Bodyko's method of combination (Bodyko 1948, ref. Hounam 1971). This method is based on studies of the soil water balance, and the following data are needed: precipitation, potential evapotranspiration and the soil water content available to plants. The latter parameter is difficult to estimate, but calculations show that the differences in actual evapotranspiration due to differences in this parameter are small (L. Carlsson and J. Falk 1978, E. Eriksson and S. Johansson 1978). The difference in evapotranspiration is less than 10 per cent for calculations at Stensele, based on a soil water content ranging from 50 mm to 250 mm. The calculations of the evapotranspiration at Juktan are therefore based on a soil water content of 250 mm, which gives a higher value of the evapotranspiration and thereby a lower value for the available precipitation. The following equation was used (L. Carlsson and J. Falk 1978) to determine the actual evapotranspiration:

$$E = E_p \frac{M}{M_0} \quad (1)$$

where E = actual evapotranspiration (mm)
E_p = potential evapotranspiration (mm)
M_0 = maximum soil water content available to plants (field capacity − wilting point) (mm)
M = soil water content at the actual time (mm)

The term M was defined by means of the equation:

$$M = M_1 + \tfrac{1}{2}\Delta M \quad (2)$$

where M_1 = water content at the start of the relevant period (mm)
ΔM = change in water content during the relevant period (mm)

and ΔM is defined as:

$$\Delta M = \frac{PM_0 - E_p M_1}{M_0 + \tfrac{1}{2}E_p} \quad (3)$$

where P = precipitation (mm)

The calculations for Stensele are based on a period of one month. The result of these calculations for Juktan is presented later under "Water balance".

River discharge

Data on the river discharge in the area are available from both the Juktån and the Umeälv. These measurements were made at hydro-power plants and reservoirs in the respective rivers. The drainage of the investigation area was measured in a creek, Olbonsbäcken, in the drainage area of Lake Renbergssjön. The measurements in the Olbonbäcken are occasional so that the discharge was related to the seasonal variations registered in the Umeälv and Juktån rivers. Annual mean values for the period 1970–1978 and calculated values for the long term period 1931–1960 are shown in Table 6.

The additional water discharge between the measuring points at Umluspen and Gardikfors on R. Umeälv originates in part from the Juktan area, although deriving also from other regions, located at high altitudes. The additional discharge is therefore too high to be representative of the Juktan area. The same is true of the discharge obtained in R. Juktån. On the other hand, the discharge in Olbonbäcken is probably more accurate for the Juktan area, although these measurements are occasional and consequently beser by major errors.

The seasonal variations in river discharge obtained at Umluspen on R. Umeälv are shown in Fig. 13, the pattern being valid for all measuring points.

Water balance

The calculated values of precipitation, evapotranspiration and available precipitation are presented in Fig. 14, which also indicates the annual duration of the snow cover.

The precipitation is calculated from the measured values at the climatological stations taking the variations in altitude into account. The evapotranspiration is determined from calculated temperature values and estimated soil water content. The available precipitation represents the difference between actual precipitation and evapotranspiration. The precipitation through the winter was regarded as stored until the first month of spring when the temperature rised above 0°C. But this is a rough approximation as the melt of the snow cover is long drawn out. The accuracy of the values obtained for the available precipitation depends on the reliability of the included parameters, since both the precipitation and the evapotranspiration are approximate, the assessment of the available precipitation will become rougher. Nevertheless, the study of the water balance will allow fixing of the months in which any precipitation may be available. The magnitude of the yearly values obtained for the available precipitation are in fairly good agreement with the water discharge calculated from the Olbonbäcken (cf. table 6).

Compared with water balance studies from other areas with similar hydrometeorological conditions, this Juktan study shows a similar yearly distribution. In a study of the water balance in the representative basin Lappträsket, M. Persson (1978:17) found that in 1968–1977, the precipitation in terms of discharge tends to become available in May during the thaw, some 50 per cent of the total the discharge then takes place. At Juktan the available precipitation in May amounts to about 75 per cent of the total throughout the year. While the study at Juktan shows little precipitation available during the summer season, Persson found high values. This discrepancy may result from the technique used at Juktan, where the calculations of the evapotranspiration were made with a high soil water content to allow a minimum value of the available precipitation. Furthermore, it must be pointed out that the values included in the present study are approximate owing to the fact that all single values are calculated from data recorded elsewhere. The Lappträsket study however, verified the yearly distribution obtained at Juktan, and indicated that the values obtained are reasonable.

Groundwater recharge

General aspects of recharge

The term recharge here denotes the process whereby the water-bearing formation is supplied with water. The mechanism of the recharge depends on whether the formation is exposed to the atmosphere or covered with other more or less permeable water-containing deposits. In the first case the recharge is due to infiltration and percolation of precipitation or surface water through the unsaturated zone down to the groundwater table. The recharge to a formation covered with other water-bearing formations is mediated chiefly by leakage from the cover. The recharge to the uncovered formation depends on:

Infiltration or percolation capacity
Available precipitation

Similarly, the recharge to the covered formation is governed by:

Leakage rate

Table 6. River discharge (in mm) via the Rivers Umeälv, Juktån and Olbonbäcken for the period 1970–1978.

Measuring station	1970	1971	1972	1973	1974	1975	1976	1977	1978	1931–1960
Umluspen	512	758	820	877	659	744	692	654	701	744
Gardikfors	612	914	1009	1111	797	921	870	797	848	928
The area between Umlupsen and Gardikfors	323	470	470	444	403	417	363	390	430	404
Juktån	461	573	588	666	488	564	386	578	551	616
Olbonbäcken	216	319	345	369	277	313	291	275	295	313

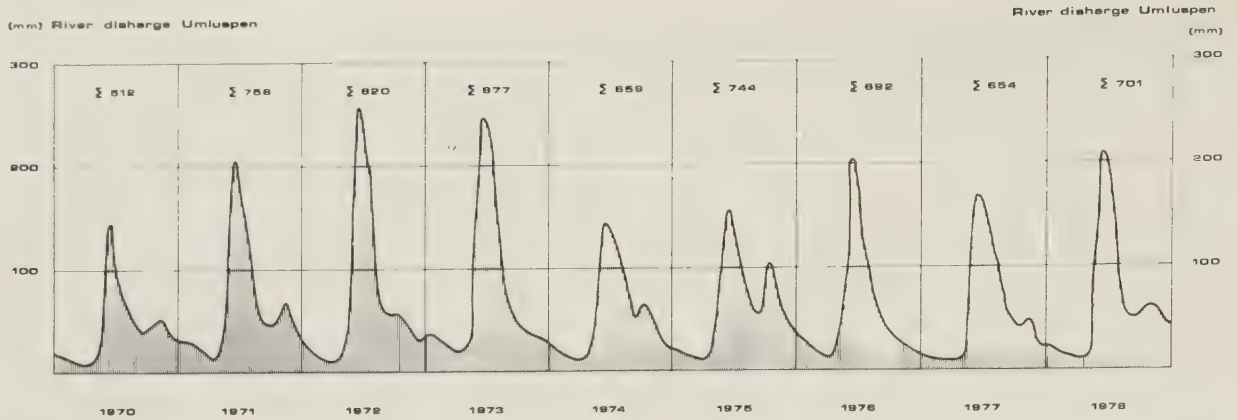


Fig. 13. Seasonal variations in river discharge at Umluspen hydro-power plant on River Umeälv. The discharge is calculated from the influx to the regulation reservoir in Lake Storuman.

Hence, the recharge to the uncovered formation depends on both hydrometeorological and hydrogeological conditions, while in covered formations it is mainly controlled by hydrogeological parameters. In terms of rise in the groundwater level, the response is governed by the effective porosity or the storage coefficient of the formation.

The values obtained for the available precipitation denote the potential for groundwater recharge. The available precipitation which percolates to the aquifer is calculated from records of fluctuations in the groundwater level at a station in Stensele.

The groundwater recharge in the aquifer is crucial for the groundwater balance. According to Brown *et al.* (1972:1.3) the long-term average discharge from a groundwater reservoir is presumably in equilibrium with the recharge. The potential recharge to the groundwater régime in an area depends on the amount and pattern of annual precipitation in relation to the evapotranspiration and the occurrence of surface or subsurface inflow from adjacent areas. The amount actually contributed to groundwater recharge varies from season to season and year to year (Brown *et al.* 1972:1.3).

In northern Sweden the groundwater recharge on the whole coincides with the spring thaw. This is shown by Nordberg and Persson (1976:6) from measurements of the seasonal fluctuations at a number of groundwater stations. Groundwater recharge may also occur in autumn in connection with heavy precipitaiton. Summer precipitation is rarely available for the groundwater recharge because of a high rate of evapotranspiration.

During the winter season almost no recharge takes place and the aquifer is continuously drained by natural groundwater discharge. The groundwater level reaches its minimum just before the thaw, but recharge recurs afterwards.

These principles for the seasonal variations in groundwater recharge demonstrate a close correlation between the recharge and the local hydrometeorological conditions (cf. Fig. 15).

Groundwater recharge in the till aquifer

No accurate method exists to determine the proportion of the available precipitation which actually contributes to the groundwater recharge. Investigations of the groundwater recharge in soil aquifers were carried out by von Brömssen (1968). He presents the recharge as a function of the annual precipitation defined by infiltration coefficients of different types of soil. These infiltration coefficients are mainly related to the soil type, while the climatological and topographical conditions were ignored. Table 7 shows infiltration coefficients based on pumping tests for different soil types according to von Brömssen (1968:105).

An acceptable method of determining the groundwater recharge analyses the fluctuations in an aquifer with respect to the actual precipitation. The observations carried out by the Geological Survey in existing groundwater stations may serve this purpose.

One of these groundwater stations is situated in Stensele close to the Juktan area (Fig. 1) and recording of various hydrogeological data is continuously in progress. The groundwater levels in till and gravel, the snow cover and the soil freezing conditions are the parameters recorded. Continuous records exist since 1971 and the station is presented in Nordberg and Persson (1974). Figure 15 shows the records for the period 1971–1978. The calculated values for the available precipitation as a source of groundwater recharge are also included.

As regards the recharge, the significance of the records consists in the response of the groundwater system to a specific amount of precipitation. The response in the aquifers varies greatly, primarily because of precipitation, soil water conditions and evapotranspiration. The intensity and duration of precipitation too affect the aquifer response (Bruce and Clark 1966:41).

Table 7. Infiltration coefficients obtained for different soil types (von Brömssen 1968).

Soil type	Infiltration coefficient
Clay – open rock – moraine area	0.11
Clay – open rock – moraine area. Permeable material under part of clay	0.16–0.37
Eskers	0.34–0.92

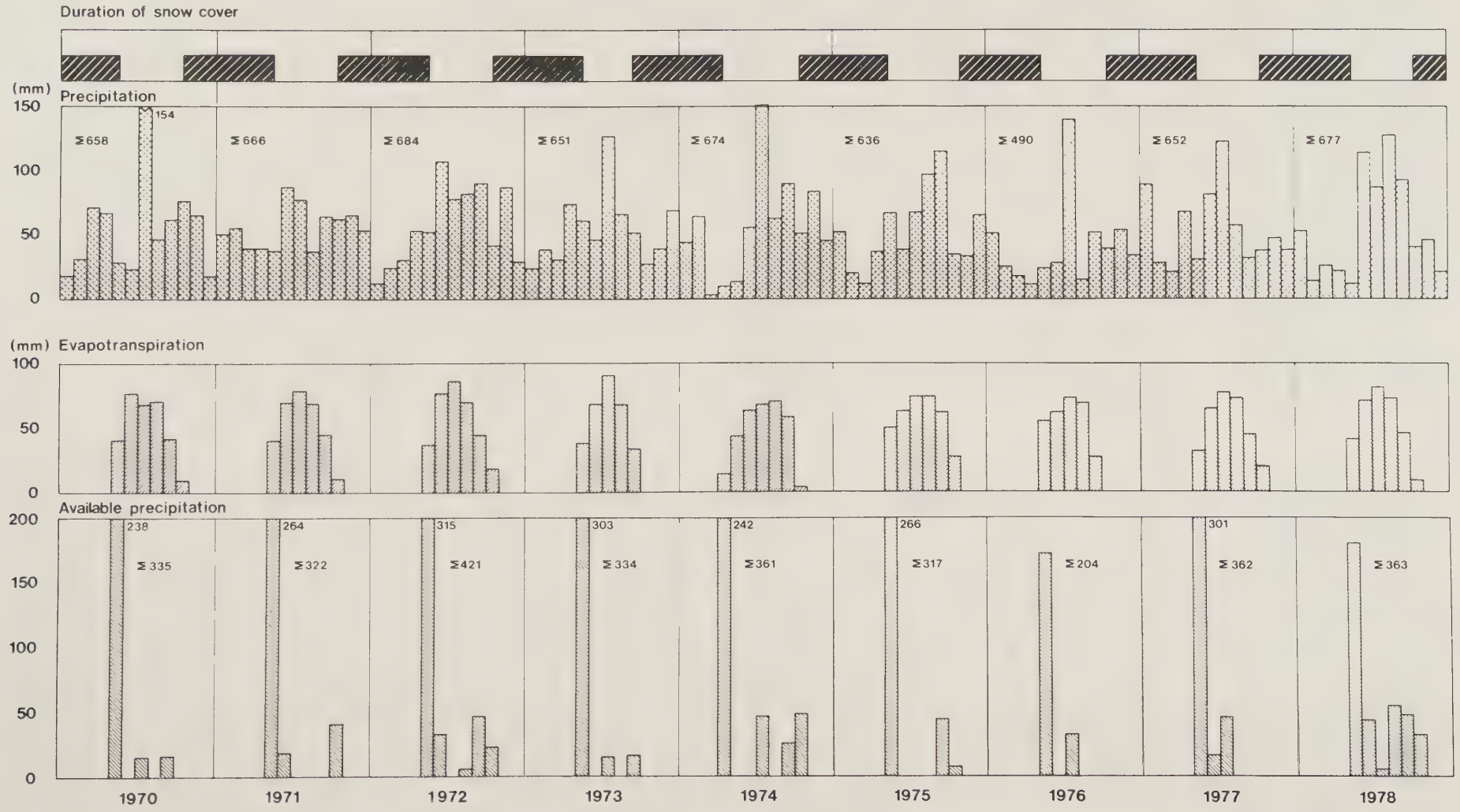


Fig. 14. Water balance of the Juktan area.

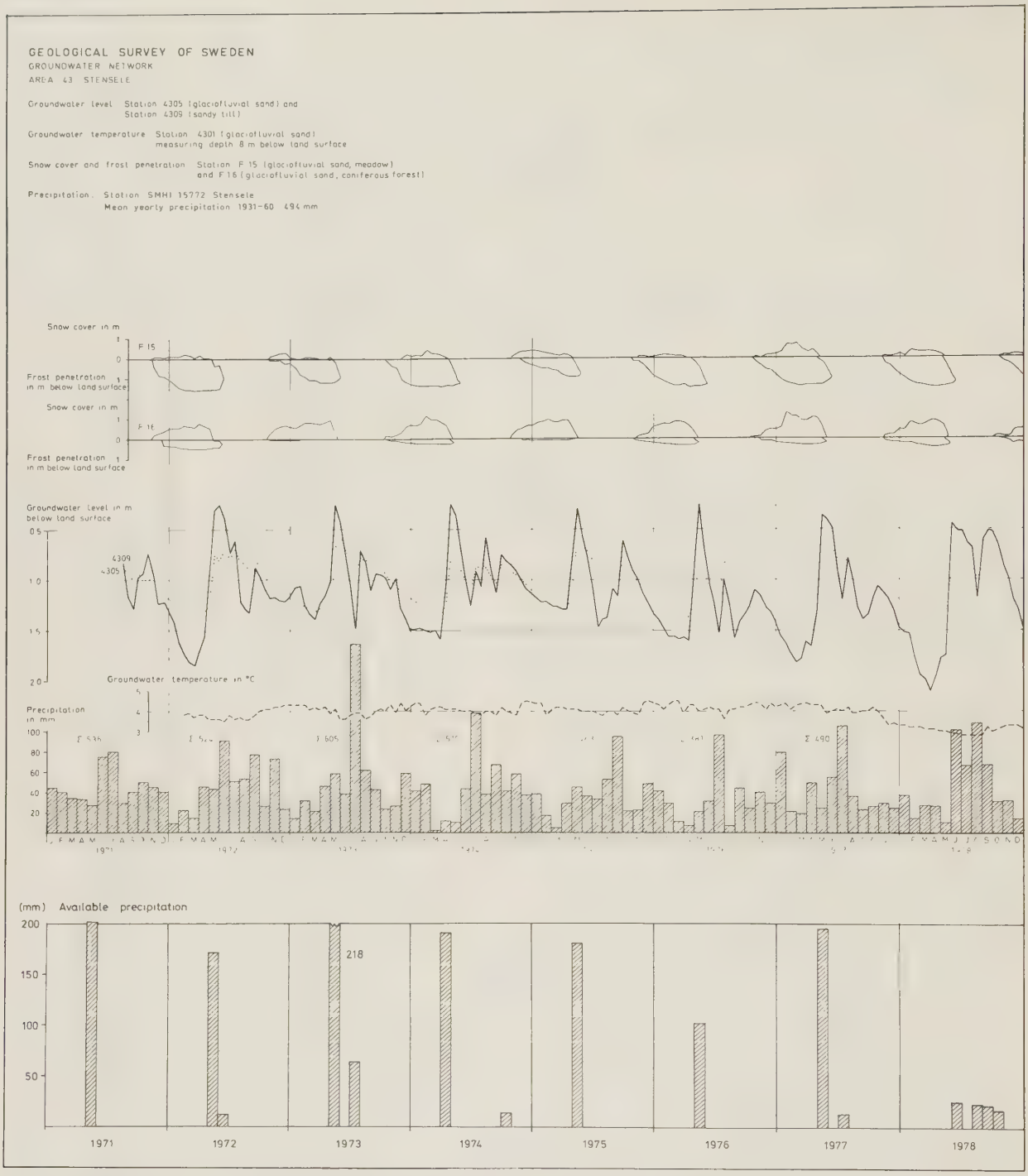


Fig. 15. Recordings of the groundwater level fluctuations, precipitation, avialable precipitation, snow cover, and frost penetration in Stensele during the period 1971–1978. Modified after Geological Survey of Sweden, Groundwater network.

The variations are caused mainly by the hydraulic properties of the aquifer. In an extensive formation with high hydraulic conductivity and porosity, the response will be levelled by means of fast pressure release within the aquifer. In a till formation with lower conductivity the pressure release will be much slower, producing a more direct reaction to the precipitation (cf. Brown *et al.* 1972:2.3.3).

The measurements taken at Stensele allow observation of the following conditions in the groundwater recharge in the till aquifer assuming that the rise in the groundwater level is thereby caused:

1. The main groundwater recharge occurs during the spring thaw. During the relevant period, 1971–1978, the aquifer reached its lowest annual groundwater level just before, and its highest level during, the melting period.

2. No considerable recharge is recorded during the summer despite high precipitation rates. Nevertheless, recharge did take place during a few summer periods due to extremely high precipitation.
3. A rise in the groundwater level usually occurs in connection with autumn precipitation, during the months August and September.
4. As a rule, no recharge takes place during the winter (October–March), on the contrary the aquifer is gradually drained.

It is possible to estimate the groundwater recharge by analysing the response in the aquifer on specific occasion of precipitation.

The potential for recharge is composed of the available precipitation. This is distributed over the main hydrological systems: surface run off, soilwater and groundwater. Surface runoff may occur when the amount of the available precipitation is higher than the infiltration capacity of the ground, and groundwater recharge happens when the soil is saturated.

In order to obtain the amount of the precipitation available for groundwater recharge and for surface runoff, a water balance study was made of the soil. The method is described by L. Carlsson and J. Falk (1978:174) and based on Budyko's method of combination (Bodyko 1948, cf. the calculations of the evapotranspiration above). The values obtained for the available precipitation in different months during the period 1971–1978 are shown in Fig. 15. Since precipitation in the months October–April falls mainly as snow it is not immediately available. This accumulated precipitation will not be available for the hydrological cycle until the spring thaw. In the calculations the thaw was assumed to take place in the first month of spring in which the temperature rises above 0°C.

As shown in Fig. 15 water is available for recharge and runoff mainly in April, (August), September and October. The fluctuations in groundwater level also indicate that the major part of the recharge usually takes place in these months. The registered hydrogeological and hydrometeorological conditions during the recharge period may therefore be used to evaluate the amount of recharge based on the aquifer response.

The most important occasion of groundwater recharge coincides with the melting of the stored snow, as demonstrated by the groundwater records in Fig. 15. Depending on the rate of melt, a variable proportion of the water will contribute to the recharge. If the thaw is slow and the ground not frozen most of the water will infiltrate (Hallgren and Sandsborg 1968:91). But, if the thaw is rapid or the ground frozen, the available amount of water may exceed the infiltration capacity of the ground. The major part of the melt-water will then be discharged as surface runoff. These conditions indicate that the groundwater response may differ due to variations in the rate of melting and frost penetration.

During the second period with available precipitation – August–October – the groundwater recharge is directly dependent on the amount of water available. In this case the response of the aquifer will probably be less ambiguous.

Brown *et al.* (1972:5.2.3) define the groundwater balance by the following equation:

$$\mu \Delta H = \Delta Q \, \Delta t + P \Delta t \qquad (4)$$

where μ = specific yield of the soil (m/m)
ΔH = rise in groundwater level (m)
ΔQ = difference in groundwater recharge and discharge from and to adjacent areas (m/s)
Δt = computed interval (s)
P = rate of groundwater recharge from above (m/s)

However, if the recharge from, and the discharge to adjacent areas are in equilibrium, and if an individual recharge occasion is computed, the equation will become.

$$P = \mu \Delta H \qquad (5)$$

here P = total recharge (m)

The total recharge during a given period, e.g. throughout the year, may be defined as:

$$P_t = \Sigma \, P_i \qquad (6)$$

where P_t = total recharge during an interval consisting of n recharge occasions (m)
P_i = total recharge on each occasion (m)

On each occasion of recharge the amount of the recharge is limited by either the amount of the available precipitation or by the infiltration capacity of the soil. For a given soil profile, the specific yield varies with time, water level, temperature and barometric pressure as pointed out by Bear *et al.* (1967:49). But, as the specific yield in an unconfined aquifer is equal to the effective porosity, the rise in the groundwater level will also be a function of the effective porosity of the soil, provided that the recharge occurs instantaneously. If the recharge is prolonged, the response may also be influenced by the discharge from the aquifer.

Analysis of the fluctuations of the groundwater level recorded at the groundwater station at Stensele provided a rough approximation of the recharge in the till aquifer. A maximum value of the effective porosity was also obtained. The discussions above, imply the validity of the following equation (cf. equation 5).

$$\Delta h_T = \frac{P_T}{\Theta_T} \qquad (7)$$

where Δh_T = rise in groundwater level (m)
P_T = groundwater recharge in the till aquifer (m)
Θ_T = effective porosity of the till (1)

In other words, the rise in groundwater level is directly due to the groundwater recharge. To diminish the effect of prolonged precipitation and melting snow during the winter, the calculations are based on single, monthly precipitations and a corresponding clear-cut response. It was not possible properly to analyse the groundwater recharge during the summer season, owing to uncertainty concerning the actual soil water conditions, and difficulties in determining the evapotranspiration. Consequently the determinations are based on data from the spring thaw and the autumn precipitation.

In the determinations, individual rises in groundwater level were analysed with respect to the available precipitation. The calculations comprise two separate steps.

Determination of the effective porosity
Determination of the recharge

In the first step the monthly amount of available precipitation on each recharge occasion was attributed to the recorded rise in groundwater level, which resulted in divergent values for the effective porosity. Of these different values, the smallest was assumed to represent the actual soil conditions, while the higher values were

Table 8. Groundwater recharge and effective porosity of the till aquifer at Stensele.

Season	Recharge (per cent of available precipitation) Mean	Standard deviation	Effective porosity per cent
Spring	60	±20	4
Autumn	70	±15	

regarded as increased via prolonged recharge and subsequent groundwater discharge. The recorded value of the effective porosity was then used in the second step where the recharge was determined. The annual recharge was determined as the sum of the individual recharge occasions.

It must be noted that determinations based on monthly values to a great extent are inaccurate because the short term variations are ignored and the amount of available precipitation depends on the character of each rainfall. These conditions are disregarded in the calculations.

The mean values obtained for the groundwater recharge in per cent of the available precipitation are shown in Table 8, together with the value of the effective porosity of the till aquifer. In general, the proportion of the available precipitation which contributes to the recharge is smaller during the spring thaw than in autumn. The variation in recharge from year to year is also greater in the values based on the spring thaw. These conditions as regards the recharge in different seasons verify the aforesaid assumptions, namely that the recharge in spring is dependent on the rate of the melt and the frost penetration.

In all, the groundwater recharge shown in Table 8 amounts on average to about 115 mm/year for the period 1971–1978, which is 60–70 per cent of the total available precipitation in the Stensele area, and 20–30 per cent of the total annual precipitation. The effective porosity is determined as about 4 per cent.

The determined values for the groundwater recharge and effective porosity are based on the Stensele observations. In the Juktan area with its higher precipitation and lower evapotranspiration, the available precipitation will be greater than the reported values. Hence, the recharge may also be higher. von Brömssen (1968) studied the formation of groundwater by rainfall and compared the annual precipitation with the groundwater discharge due to pumping tests in water wells. These studies were carried out in small drainage-precipitation areas and the results provide infiltration coefficients of about 11 per cent for till covered areas (von Brömssen 1968:105). However the total area of the drainage basins minus the area of clay and marshy ground is presumably influenced by the pumping. This assumption is probably incorrect. The investigated drainage basins, $4–6 \cdot 10^5$ m^2 are too large an area to be influenced by pumping tests in till soils (cf. Engqvist and Olsson 1974:32). The infiltration rate within the affected area must therefore be higher, as evidenced by measurements of the infiltration capacity in till deposits. Measurements executed by von Brömssen (1968:84) involve stationary infiltration rates of between 3–11 mm/min, which indicate that the infiltration potential is much higher than the values obtained for the infiltration coefficients. Yet, von Brömssen ignored the recorded infiltration capacities when he determined his infiltration coefficients. In the Juktan area a few simple tests of the infiltration capacities of the till deposits were performed by means of infiltrometers alike that method described by von Brömssen (1968). These tests were made in the vicinity of Lake Blaiksjön and at the intermediate access tunnel, the average infiltration rate being 3–5 mm/min, i.e. the same order of magnitude as the values obtained by von Brömssen. The infiltration capacities versus time are shown in Fig. 16.

The measurements in the Juktan area, like those of von

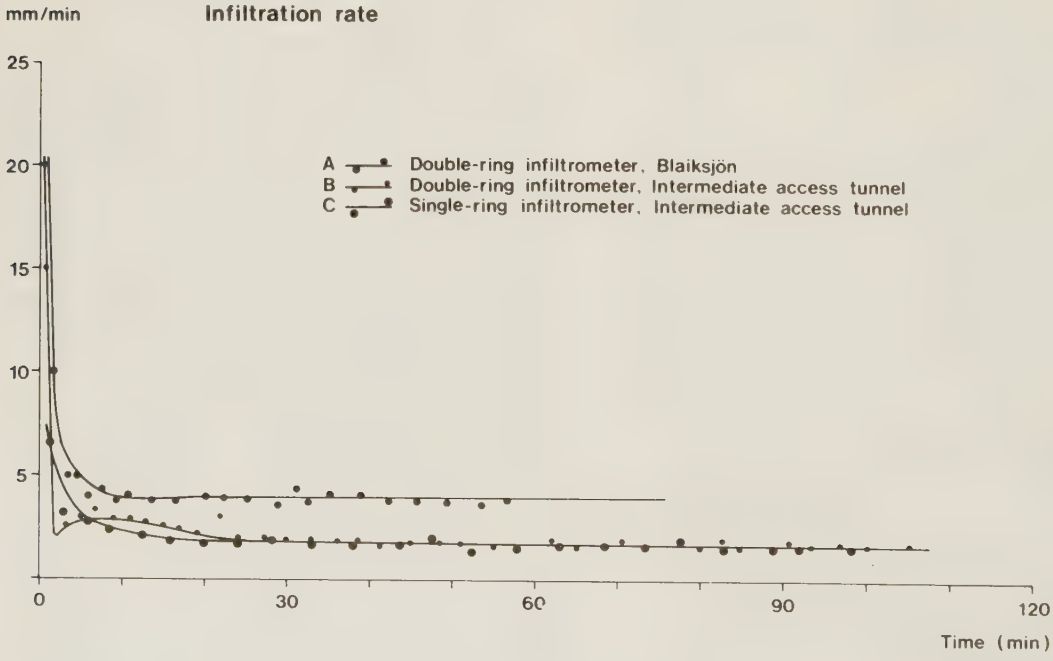


Fig. 16. Infiltration capacity versus time of the till soils in the area of Lake Blaiksjön, and at the intermediate access tunnel.

Brömssen, were taken at the ground surface. According to Wedel (1978:75), the infiltration capacity is much higher at the ground surface than lower down. Wedel found a considerable difference between the surface and a depth of 30 cm, but these measurements refer to clay soils. This effect is probably due to higher conductivity and higher porosity as shown by Lundin (1977:30). In investigations in till soils Lundin found that the porosity decreases with depth, from about 60 per cent in the upper 10 cm to 20 per cent at a depth of 1 m. Moreover, the hydraulic conductivity diminishes rapidly with increasing depth. Lundin (1978:32) reports horizontal groundwater velocities in the saturated zone of 0.18 m/day and 0.06 m/day at depths of 0.4 m and 1 m respectively, with more or less equal hydraulic gradients.

The infiltration rates obtained in till are so high as to suggest that all available precipitation infiltrates and shares in the groundwater recharge. This circumstance was also pointed out by Wedel (1978:78).

Groundwater recharge in the rock aquifer

Bergman (1972) studied the infiltration in exposed rocks, and found percolation rates of between 0.2–4.4 mm/h in gneiss and 2.8–6.3 mm/h in granite. The infiltration potential is thus limited in rock compared with till formations. Bergman (1974:29) found that the recharge into rock depends to a great extent on the existence of soil cover. It was also discovered that the inclination of the rock surface is crucial for the amount of recharge. The recharge is greater on flat than on inclined rock surfaces due to decreased surface run off, provided that the available precipitation is greater than the infiltration rate. In the former case the recharge is governed by the presence of open fractures at the rock surface which the available water can penetrate. This infiltration only takes place during rainfall. In the latter case the soil cover acts as an infiltration reservoir, multiplying the potential for groundwater recharge. The amount of the recharge is dependent on differences in hydraulic head between the soil cover and the rock mass, and on the conductivity of the formations.

As the rock mass in the Juktan area is covered by till deposits, it seems relevant to regard the recharge to the rock aquifer as a function of the recharge to the till. The groundwater levels in the rock mass and the till are assumed to be the same when the systems are in equilibrium. However groundwater recharge in the till raises the level of the table creating a potential gradient between the aquifers. The obtained difference in hydraulic head induces a flow of groundwater from the till to the rock mass, whereupon the equilibrium is restored. This induced groundwater flow is the actual groundwater recharge to the rock aquifer and its magnitude depends on the effective porosity or storage coefficient of the till and the rock mass, and on the recharge to the till aquifer. With a instantaneous recharge to the till, the following equation is assumed valid for the till aquifer (equation 7).

$$\Delta h_T = \frac{P_T}{\Theta_T} \tag{7}$$

where Δh_T = rise in groundwater level in the till (m)
P_T = groundwater recharge to the till (m)
Θ_T = effective porosity of the till (1)

Assuming that this rise in groundwater level in the till aquifer is compensated by a corresponding rise in the piezometric head in the rock mass. By analogy, the response in the rock aquifer can be written:

$$\Delta h_R = \frac{P_R}{\Theta_R} \tag{8}$$

where Δh_R = rise in piezometric head in the rock mass (m)
P_R = groundwater recharge to the rock mass (m)
Θ_R = storage coefficient of the rock mass (1)

Assuming $\Delta h_T = \Delta h_R$ the equations may be combined, which gives

$$P_R = P_T \frac{\Theta_R}{\Theta_T} \tag{9}$$

This equation is graphically expressed in Fig. 17.

The superficial rock mass in the Juktan area has a kinematic porosity estimated to about $3 \cdot 10^{-3}$ (cf. below "Groundwater in the rock mass"), which gives a recharge of about 7 mm/year. This is based on the calculations from Stensele, and is valid insofar as the conditions at Stensele and Juktan are identical.

Discussion of the recharge values

The calculations of the recharge to the till soil and the rock mass must be considered as rough approximations. As regards the recharge to the till soil they are fairly accurate provided the determination of the available precipitation holds good. A treatment of the available precipitation and groundwater level response in a shorter time scale, based on days, would probably yield a more reliable result. Nevertheless, the result shows that the majority of the available precipitation takes part in the recharge process, which is in agreement with results presented by E. Eriksson (1974). This circumstance is also indicated by the infiltration capacities obtained for the tills in the area.

The value obtained for the recharge in the rock mass, 7 mm/year, seems accurate compared with reported values of infiltration capacities in rock masses. As regards the equations used for the determination, it is a moot point whether the effective porosity or the elastic storage coefficient should be used. The use of the storage coefficient would yield a lower recharge to the rock mass than the value presented, which is based on the kinematic porosity.

Groundwater in the Quaternary deposits

At Juktan the till is the most extensive Quaternary deposit, although large areas are covered by a thin layer of peat. Due to its extent, the till together with the basement rocks, forms the major aquifers. The peat bogs may be of great importance for the groundwater system, primarily because of their high water content. Limited extent, but high infiltration and storage capacity of the sand and gravel deposits indicate that these formations are of local significance. However, this section will concentrate on the groundwater conditions in the till soils.

Hydraulic properties of the till soils

Determinations of hydraulic conductivity of till soils carried out in the laboratory (Fagerström and Wiesel

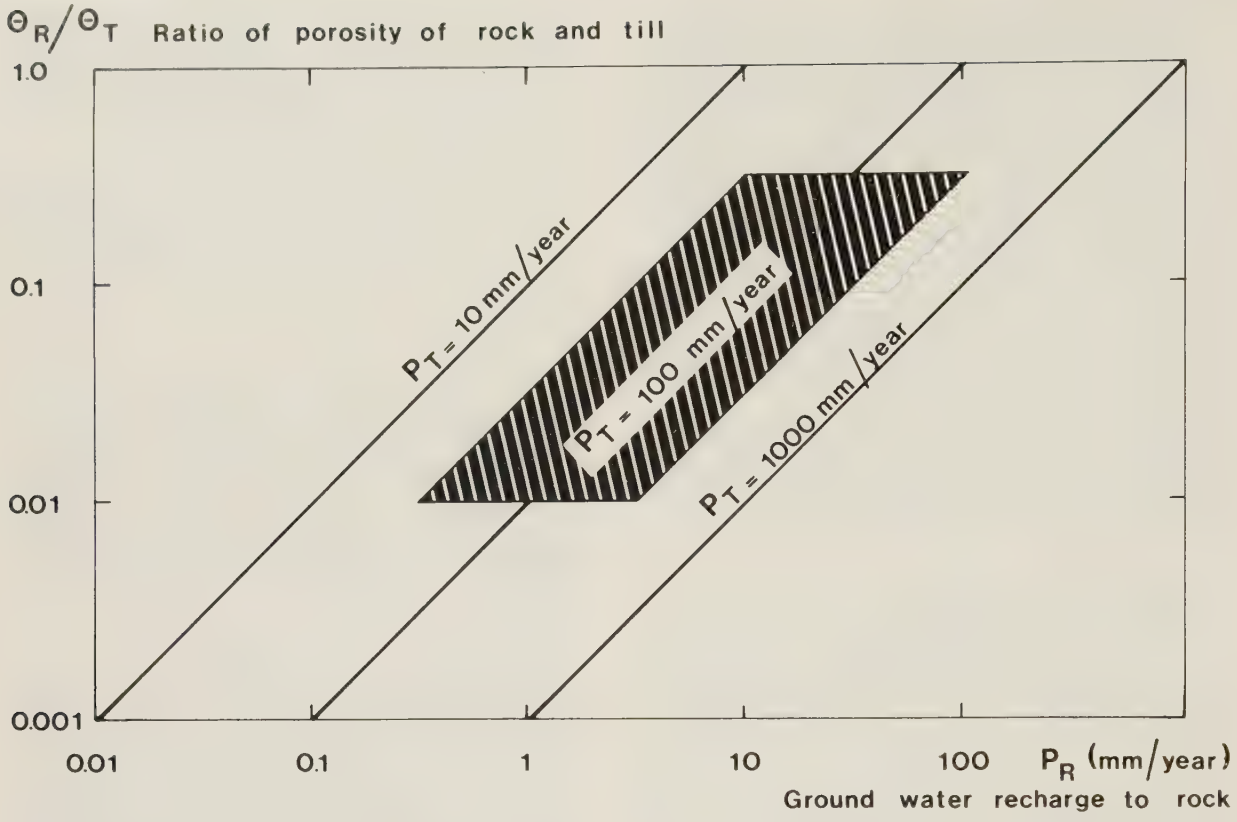


Fig. 17. Relation between groundwater recharge in the till and rock aquifers as a function of the differences in effective porosity.

1972:9), with disturbed samples give values ranging from $5 \cdot 10^{-7}$ m/s in sandy-gravelly till down to $5 \cdot 10^{-11}$ m/s in clayey-silty till (Fig. 18). A sediment of silty-sandy composition has a conductivity of about 10^{-6} m/s, while a till of similar content, i.e. a silty-sandy till, has a conductivity of about 10^{-9} m/s. This difference is primarily due to varying degrees of grading between the sorted sediment and the unsorted till.

In situ determinations of the hydraulic conductivity or water-yielding capacity of till soils are reported in the Swedish literature by inter alios Knutsson (1966 and 1971), Engqvist and Olsson (1974), Andersson and Svensson (1975) and Engqvist *et al.* (1978). Differences in the methods used for the determination complicate comparison of the reported results. Knutsson carried out short-term pumping tests in wells or springs situated in different types of moraine. Engqvist and Olsson and Engqvist *et al.* measured the recovery rate in till wells after instantaneous pumping. Andersson and Svensson have carried out an infiltration test in pipes in a till formation.

In those works in which conductivity values are reported (Engqvist and Olsson 1974:33, Andersson and Svensson 1975:12 and Engqvist *et al.* 1978:I–48), the results of the *in situ* determinations diverge from expectations with regard to the granulometric composition of the till. The *in situ* tests usually seem to give conductivity values which are more than one decade higher than those obtained in laboratory tests. The investigation reported by Andersson and Svensson (1975:12) were performed in a gravelly-sandy till, which judging by its composition would have a hydraulic conductivity of about 10^{-7} m/s (Fagerström and Wiesel 1972). The results obtained at *in situ* determination give a conductivity of about $5 \cdot 10^{-6}$ m/s. In principle, the same applies to the results reported by Engqvist and Olsson (1974) and Engqvist *et al.* (1978). Due to the unsorted composition of the till soils, its effective porosity is small. Values of about 5 per cent are commonly reported in the literature.

Hydraulic conductivity of the till deposits

According to Fagerström and Wiesel (1972:8) the following parameters are of importance for conductivity of soils.

Grain size distribution and grading
Stratification and fissility
Rate of compaction
Porosity
Air content of the water

These parameters relate to saturated conditions. In unsaturated conditions, the water content of the soil is an additional parameter of significance. Of the listed parameters, the first four are directly attributed to the soil material as constants depending on the interior structure of the soil.

Till fabric. The Swedish literature dealing with the fabric of till deposits mentioned the occurrence of discontinuities in

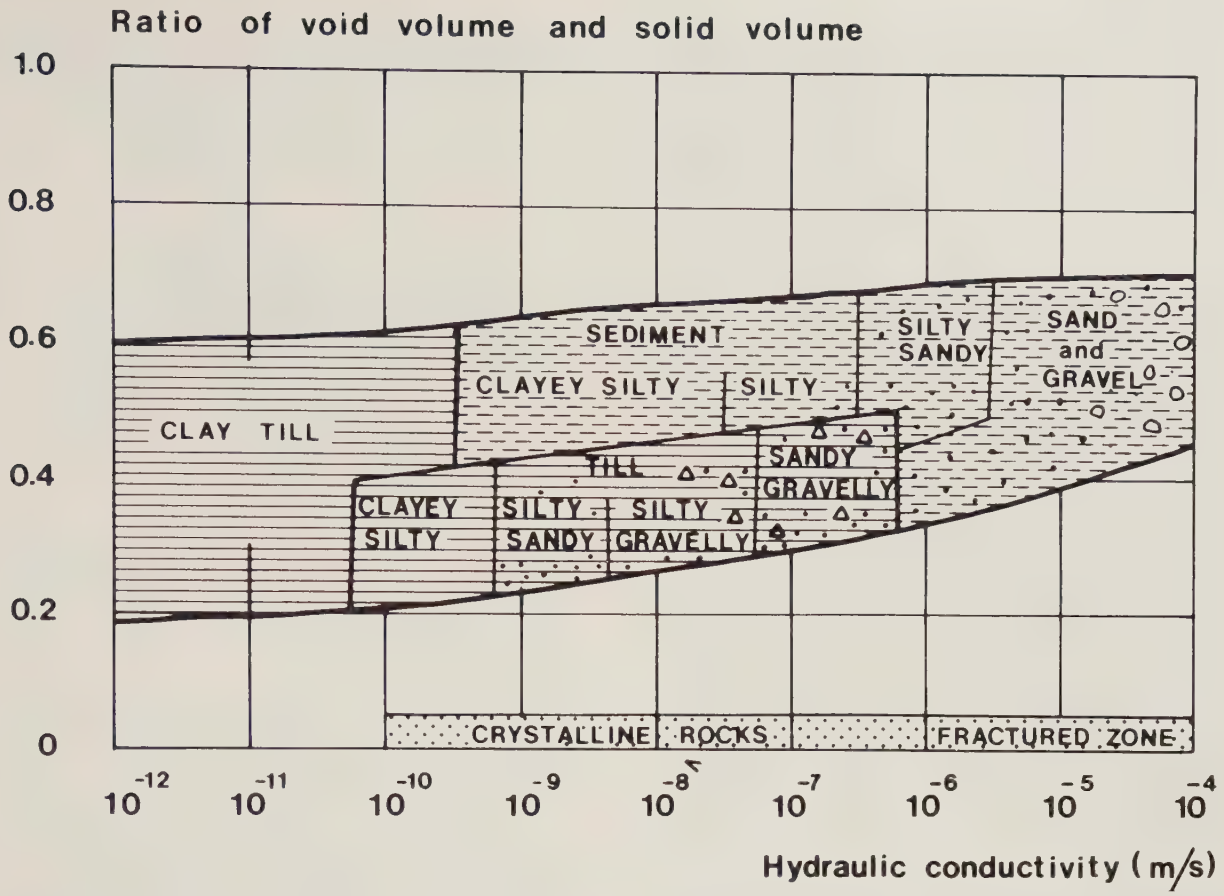


Fig. 18. Laboratory values of the hydraulic conductivity and porosity of Quaternary soil types and crystalline rock types. Modified after Fagerström and Wiesel (1972).

these formations (G. Lundqvist 1940, J. Lundqvist 1968, Minell 1979 and others) and proved lenses and thin strata of sorted material to be a common feature, a view supported by several published geological descriptions. These discontinuities are usually limited in lateral extent although prolonged lenses do occur (Knutsson and Morfeldt 1973:61). The material in the lenses is usually sand or silt as reported by *inter alios* G. Lundqvist (1940:30) and J. Lundqvist (1968:56). Möller (1960:185) among others found that till containing lenses normally lies directly on the underlying bedrock, being overlain in places by hard, packed and pressed till. According to Minell (1979:15) the sediment-bearing till is generally sandy with a clay content of 2 per cent or less.

G. Lundqvist (1940:31) discovered that till deposits are often foliated, and various subgroups can be identified such as schistose, varved, sheeted or brecciated till. In general horizontal orientation of the structures predominates. According to J. Lundqvist (1968:55) the fissility can only be observed under certain circumstances such as differences in colour due to differences in soil moisture. Groundwater occurs in some layers while others are completely dry, an effect which recurs in the zone of aeration or after drainage.

These structural discontinuities in an otherwise homogeneous till matrix, may cause a considerable increase locally in hydraulic conducitivity as compared with an intact till. As alluded in the account of the Juktan area, the till deposits tend to be structured and lenses of sorted material as well as fissility are common.

As seen from Fig. 18, laboratory determinations of the hydraulic conductivity of till material range from about 10^{-11} to $5 \cdot 10^{-7}$ m/s depending on the granulometric composition. The existence of discontinuities in a natural till formation may substantially increase the conductivity. The importance of the discontinuities is illustrated by the following example:

A till deposit of silty-sandy composition (conductivity 10^{-9} m/s) is intersected by a sorted lens of silty sandy composition (conductivity 10^{-6} m/s). With a total thickness of the deposit of 5 m, of which the lens represents 0.1 m, the combined conductivity of the deposit can be determined as (cf. Terzaghi and Peck 1967):

$$k = \frac{1}{5}(4.9 \cdot 10^{-9} + 0.1 \cdot 10^{-6}) = 2 \cdot 10^{-8} \text{ m/s}$$

Thus the effect exerted on the conductivity by the presence of the lens is considerable and the combined conductivity is about 20 times greater than that of the till matrix. However, this is only valid at the local level, in the regional scale it also is necessary that the lenses are interconnected. A similar effect may be caused by stratification and fissility, but is probably of a smaller magnitude.

The importance of the till fabric is emphasized by e.g. Knutsson (1966:248 and 1971:13), Williams and Farvolden (1967:169) and Olsson (1977:13). Knutsson

(1971:13) discovered a distinct relationship between the structure and the stratigraphy of the till soils, while the grain size distribution is apparently less significant. A comparison between two moraine types–drumlin type and hummocky moraine–shows that the till in the drumlin terrain usually has the higher conductivity, a difference due to differences in morphology, structure and stratigraphy. In general, the till formations in the drumlin terrain contain interconnected, stratified discontinuities, while those present in the hummocky moraine are more or less contorted. Nevertheless the frequency of the lenses is higher in hummocky moraine. But investigations carried out by Kunkle (1962:16) in clayey till in Canada demonstrate the reverse situation. He reports a higher degree of hydraulic interconnection in hummocky moraine than in hard, packed basal till. Williams and Farvolden (1967:166) point out that the permeability of joints in glacial till is far greater than that of intergranular pore spaces in till. The importance of fractures in till is also underlined by Day (1977) and Cherry *et al.* (1979).

The existence of directed structures within a till formation indicates dispersion of the hydraulic conductivity. In general, the horizontal conductivity would exceed the vertical due to a predominance of horizontal structures. In the superficial deposits, the presence shrinkage fractures and biological activity would also cause a higher vertical conductivity. These conditions are illustrated by a higher infiltration capacity at the ground surface (Wedel 1978:75, in clay soils) and by greater conductivity and porosity (Lundin 1977:30, 32).

Determination of hydraulic conductivity. Till material from the vicinity of Lake Blaiksjön was used in the rock-fill dams built to regulate the lake. The conductivity of the till was tested in this context. The conductivity determinations were performed by the staff at Juktan on disturbed samples by means of simple permeameter tests.

The permeameters consist of a cylinder to which a bottom plate and a lid are attached. The bottom plate is welded to the cylinder, 150 mm in diameter by 250 mm high, and the plate fitted with a drain pipe. Both lid and bottom plate are of a greater diameter than the cylinder, which allows the lid to be attached by three bolts outside the cylinder. A connection for tubes is inserted in the lid through which the permeameter is supplied with water of constant head. A drainage layer of sorted gravel is placed in the permeameter to a height of about 30 mm, then covered with a wire net. Till material is placed in the cylinder and compacted by a modified proctor compaction (Bernell and Löfquist 1958:85) with 50 strokes at each layer (altogether 4 layers of about 50 mm). The total height of the till sample in the permeameter is about 200 mm. In order to avoid a higher porosity near the walls of the cylinder, a thin layer of asphalt is attached. Above the compacted till another layer of sorted gravel is placed. Both this and the layer underneath the till sample cause a release of water pressure and a constant head acting throughout the sample.

At the start, the permeameter is supplied with water of constant head, and the percolating water is collected in a vessel at the end of the drain pipe. Normally several permeameters are in operation at the same time and Fig. 19 shows their arrangement. After a period of saturation,

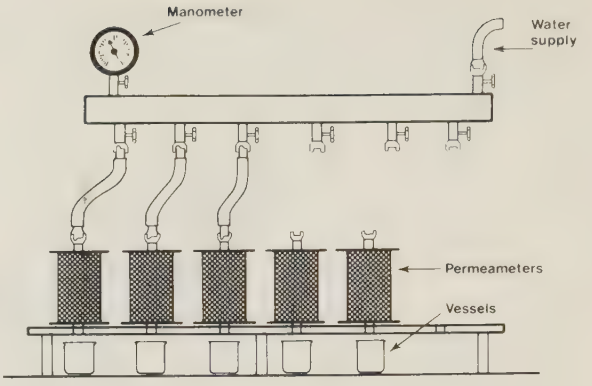


Fig. 19. Arrangement of the permeameters during testing. Modified after Bernell and Löfqvist (1958).

the actual measurement begins by measurement of the volume of water which has percolated through the sample in a given time. The hydraulic conductivity of the till sample is obtained by Darcy's law.

$$Q = kA\frac{dh}{dl} \qquad (10)$$

where Q = volume of water passing through the sample in unit time (m^3/s)
A = area of the sample (m^3)
k = hydraulic conductivity of the sample (m/s)
dh/dl = hydraulic gradient, change in head (h) per unit length (l) along the flow path (m/m)

In all, 149 determinations of the hydraulic conductivity were made, the distribution of the results being shown in Fig. 20. Mean values, standard deviation and median values are given in Table 9. All values are similar and gathered within half a log unit.

The till material was disturbed during sampling and preparation, which destroyed the original stratification and thus, excluded the effects of the discontinuities on the conductivity. This difference between disturbed and undisturbed samples was pointed out by Kopecky (1914:175) who regards the measurements with disturbed samples as incorrect, because the structure of the soil after packing diverges from that of the natural soil. The discrepancy between results obtained from disturbed and undisturbed samples depends on the similarity between the disturbed sample and the natural till soil.

The differences between *in situ* determinations and laboratory tests are due to the till fabric and the presence of structural discontinuities, which cause a greater conductivity in natural, stratified till. The effects of these discontinuities can only be discerned after *in situ* tests or measurements with undisturbed samples. Determinations carried out in the laboratory with disturbed samples will represent the till matrix excluding the discontinuities rather than the natural till.

According to Knutsson (1971:13), the grain size distribution, packing etc. of the till do not seem to be as significant for the groundwater conditions as the structure and stratigraphy of the till soil. However, the effect of the structure is a matter of scale. In the regional scale the degree of interconnection limits the total effect of the structural discontinuities. The water flow through the sample during testing is directed parallel to the direction of the

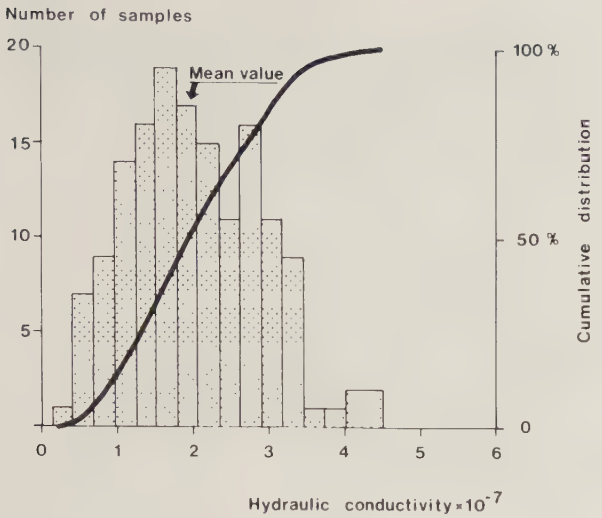


Fig. 20. The distribution of the values obtained for the hydraulic conductivity of the till material.

packing force during compaction. In the natural till, this force corresponds to the static and dynamic compression of the ice cap. The conductivity values obtained are therefore probably more representative of the vertical than the horizontal conductivity.

Airfilled voids within the sample decrease the conductivity, and measurements made before complete saturation give an idea of the hydraulic conductivity of the natural till above the groundwater level. The measurements were of such duration, however, that the effect of airfilled voids would be slight (cf. Sillanpää 1956:25). Natural groundwater normally has a low air content and the use of water with a comparatively high air content will produce an unduly low value for the hydraulic conductivity.

According to results reported by Sillanpää (1956:102) the differences due to temperature, air content, saturation etc. fall far short of those inherent in the structure of the till material.

The conductivity values obtained for the Juktan till are, considering disturbed samples were used, probably more representative of the vertical conductivity below the groundwater level. At the ground surface the vertical conductivity is higher by reason of microbiological, chemical and physical processes. Locally, the horizontal conductivity of the till soil will be much greater than the recorded values because of the existence of predominant horizontal structures. On a regional scale, however, a greater conductivity exists insofar as the discontinuities are hydraulically interconnected. At the ground surface too the horizontal conductivity seems to be higher than at greater depth. Tamm (1940:167) found that groundwater drainage in till soils takes place primarily through a system of arteries and discontinuities, while groundwater discharge as broad flow occurs only in seasons with high groundwater levels. This broad flow ceases, when no more water recharges the aquifer, whereupon the flow resumes in the arteries. By means of tracer tests Lundin (1977:32) discovered that the majority of the groundwater flow in till soils is confined to the upper soil layer, parallel to the ground surface. The flow at 1 m depth is only about 30 per cent of that found at a depth of 0.4 m.

Table 9. Hydraulic conductivity of till material from the vicinity of Lake Blaiksjön.

Hydraulic conductivity (m/s)		Standard deviation (m/s)
Median	Mean	
$2 \cdot 10^{-7}$	$2 \cdot 10^{-7}$	$\pm 0.9 \cdot 10^{-7}$

Effective porosity of till soils. According to Todd (1959:15, 23) it is possible to distinguish a total and an effective porosity in a soil. The total porosity is defined as the ratio between void space and total volume of the mass while the effective porosity (or specific yield) is defined as the ratio between the volume of water drained by gravity and the total volume of the mass. In addition, a third type of porosity may also occur, namely the flow or kinematic porosity which is the part of the whole in which the groundwater flow occurs. The kinematic porosity depends on the hydraulic gradient, and at a hydraulic gradient equal to 1 corresponds to the effective porosity.L. Carlsson (1970:91) showed by means of pumping and tracer tests that there is a essential difference between the different types of porosity.

By reason of its unsorted composition the till soil has a low effective porosity. Nordberg and Modig (1974:332) report values for effective porosity of about 5 per cent and a total porosity of about 25 per cent. Andersson and Svensson (1975:12) recorded 0.4 per cent as the effective porosity of the same till deposit. The latter value, however, represents the storage coefficient of the till, and if this equals effective porosity, the aquifer must be unconfined. In view of the fact that the water-bearing ability of till soils depends mainly on the more or less well developed horizontal structure with limited vertical connections, the till deposits in reality function as confined or semiconfined aquifers. In this case the storage coefficient of the aquifer also depends on the compressibility of the soil. The storage coefficient is therefore lower than the effective porosity of the aquifer. Lundin (1977:31) reports values for the effective porosity of about 2.3 per cent.

In order to evaluate the effective porosity in the Juktan area, the groundwater response to precipitation was studied (cf. the former section on "Groundwater recharge"). The following relation between the recharge and the rise in groundwater level is assumed valid for an instantaneous groundwater recharge:

$$P_T = \Theta_T \, \Delta h \qquad (11)$$

where P_T = groundwater recharge in the till aquifer (m)
Θ_T = effective porosity of the till formation (l)
Δh = rise in groundwater level (m)

But if the recharge is not instantaneous, an apparently increased value of the effective porosity will be obtained, because the quantity of the discharge during the recharge period carries great weight. The total rise in the groundwater level is in this case governed by the actual recharge minus the discharge. The estimations based on the total recharge will therefore yield too high a value of the effective porosity, i.e. a maximum value.

Assuming that the total available precipitation from a given rainfall will contribute to the groundwater recharge, a fictive value of the effective porosity may be determined. The effective porosity is fictive in the sense that the

recharge is not in fact instantaneous, but a small quantity of water will invariably be discharged during the recharge period. In other words the obtained value is composed of the effective porosity plus an unknown discharge. The assumptions of instantaneous recharge and contribution of the total available precipitation will probably hold good occasionally. Consequently it is possible to obtain a fair approximation of the effective porosity by analysing a great number of increases in groundwater level. The lowest value obtained for the effective porosity will be approximately equal to the real effective porosity of the aquifer.

Groundwater and precipitation recordings at Stensele from the period 1971–1978 provided the basis of the analysis. The values obtained for the effective porosity range from about 4 to over 30 per cent. The latter value includes a considerable amount of groundwater discharge, but possibly also erroneous estimates of the available precipitation. Indeed, while the lowest value will probably correspond closely with the effective porosity of the till, it is affected by the accuracy of the estimated available precipitation. The magnitude of the effective porosity, 4 per cent, agrees however, with the values reported by Nordberg and Modig (1974) and Lundin (1977) from other till deposits.

As stated above, there is some doubt whether the effective porosity actually does govern the rise in the groundwater level due to recharge. The low values obtained by Andersson and Svensson (1975) indicate that the till aquifer acts as a confined or as semiconfined aquifer. This effect may, however, be an effect of the test method used in their study, with profound infiltration (0.5 m above the rock surface) and shallow observation (1 m below the groundwater level).

Thus it is possible that the obtained values are due to both the effective porosity and the storage coefficient. This would also indicate that the behaviour of a till aquifer may vary according to depth. The superficial till formation may function as an unconfined aquifer while lower layers may seem confined.

Groundwater in the rock mass

The water-bearing capacity of crystalline rocks derives from interconnected fractures, which applies both to the hydraulic conductivity and the storativity of the formation. The water bearing capacity varies not only between different rock types, but also in one and the same rock type. Statistical studies by Wenner (1951) and L. Carlsson and A. Carlstedt (1977) prove the existence of distinct differences between rock types, which are generally thought to depend on the disposition to brittle fracturing and the development of conductive fracture systems (Larsson 1972). The variations in one and the same rock type are due to variations in fracture characteristics for discontinuities may differ in magnitude, and of importance for the hydraulic properties of the rock.

In this context the terms rock matrix and rock mass denote rock in different scales. The rock matrix is here defined as rock material including microdiscontinuities such as joints between crystals, while the rock mass according to ISRM (1975:4) represents the rock as it occurs *in situ*, including its structural discontinuities. Thus, the rock mass consists of a great number of blocks of rock matrix separated by individual fractures. Furthermore, zones of fractured or crushed rock are used of large structural discontinuities intersecting the rock mass.

Groundwater leakage in the tunnel-system

Capacity of the groundwater inflow. The groundwater leakage in the Juktan tunnels amounts to a total 250 l/s, which gives a leak-intensity of 0.010 l/s m tunnel for the tunnels as a whole. These values are less than those of A. Carlsson and T. Olsson (1978a:379) since further drainage of the rock mass has taken place since the measurements reported in 1978. The time lapse of two of the largest water inflows may be seen in Fig. 23. The intensity value obtained for the Juktan tunnels may be compared with 0.027 l/s m, the average leak intensity calculated for the submarine discharge tunnel at the Forsmark nuclear power plant (A. Carlsson and T. Olsson 1977c:19) and 0.0005 l/s m, the intensity in the Himmerfjärd tunnels (A. Eriksson 1975).

But there is a great difference in hydraulic conditions between these tunnels. A comparison between different tunnels is shown in Table 10. In the Forsmark tunnel the rock cover is 40 m thick on average, while in the Juktan tunnels it reaches 225 m. Another major difference between the tunnels is that the possibilities of infiltration favour the Forsmark tunnel, by reason of its location beneath the Gulf of Bothnia.

Groundwater inflow related to the tectonic conditions. The highest inflows in the Juktan tunnels can be assigned to specific zones (Fig. 6 and 21), where the inflow amounts to about 200 l/s of the total capacity of 250 l/s. The intervening rock mass represents parts of the tunnel system with minor flooding and long stretches are devoid of noticeable inflows. The greatest individual groundwater inflows are reported in Table 11. These are connected with large tectonic zones presenting great differences, both as regards the tectonic and geological properties and the extent of the inflow.

The water inflows A and B (Fig. 21) in the outlet of the tailrace tunnel are due to steeply dipping tectonic zones, which are topographically indicated by the valley in which Lake Renbergssjön is situated. The rock is severely crushed within and also to some extent between these zones. The mean fracture frequency here amounts to about 10 fractures per m.

In the initial stage of the inflow in these zones the inflow capacity was about 75 l/s, but this diminished with time to a stationary flow of about 35 l/s. The decrease indicates drainage of the rock mass. The possibility of using Lake Renbergssjön as a recharge reservoir for the zones has not been realized and the infiltration rate seems to be low.

The leakage marked C (Fig. 21) in the intermediate stretch of the tailrace tunnel emanates from a flat thrust-fault which dips about 10°S. Initially, the water inflow amounted to about 85 l/s which rapidly decreased to a stationary flow of 35 l/s.

The largest initial inflow (leakage D) was as high as 180

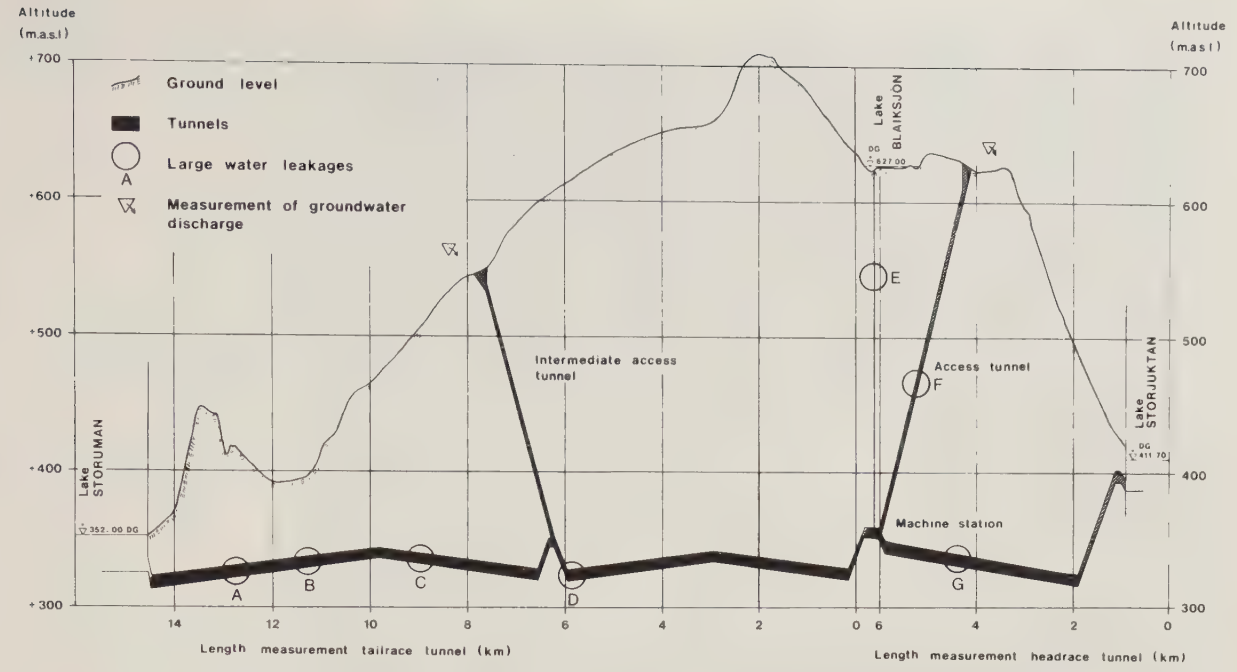


Fig. 21. Longitudinal profile through the tunnels and the corresponding altitudes of the ground surface. Areas with large water inflows are marked in accordance with the notations in Fig. 6. (A. Carlsson and T. Olsson 1978a).

l/s at first, but soon fall to and stabilized at 35 l/s. Figure 22 shows a distinct water inflow via an inclined crushed zone within this main zone.

Figure 23 shows the total inflow versus time to the stretch of the tailrace tunnel drained by pumping at the intermediate access tunnel. The flow was measured at a measuring station on the ground into which the groundwater was discharged. The large leakages C and D are clearly seen in this diagram. The decrease in inflow capacity is also visible. The delay in the inflow in zone D indicates that the rock mass is being drained and the groundwater recharge is at present in balance with the discharge.

The cable shaft in the machine station area penetrates a 20 m wide, flat zone which dips about 10°N. Initially the inflow was about 20 l/s, but the capacity was reduced by cement-grouting to 2 l/s.

Leakage F is situated in the access tunnel to the machine hall, and has the highest stationary inflow capacity of all water-bearing zones in the tunnel system. The inflow amounts to about 50 l/s and was originally about 100 l/s. This inflow occurs in a flat fracture zone dipping about 10°N. As both zone and tunnel are flat, the zone is exposed along a fairly long stretch. This zone may be identical with the zone penetrated by the cable shaft (leakage E). The geometrical orientation of both zones coincides and their extrapolation gives penetration points in agreement with those observed.

The fracture orientation in this zone is shown in Fig. 24 and 25. Figure 24 is a stereographic projection of the intersections between the lower hemisphere and the normal of the fracture planes, while Fig. 25 present the percentage distribution in the strike of all vertical and inclined fractures. According to the fracture orientation three joint sets are present, one horizontal and two vertical or subvertical. The directions of the vertical sets are N80°W and N40°E respectively, which accords with the general fracture pattern reported for the Juktan area (cf. above "Geology"). Apart from these joint sets fractures are observed which can not be attributed to any set. In all, the

Table 10. Comparison between different tunnels, regarding the conditions for water inflows.

Tunnel	Main rock type	Average rock cover m	Recharge sources	Tectonic zones, % of tunnel length	Cross-sectional area m^2	Inflow l/s m	Max. leak-intensity l/s m	Reference
Juktan	Granite	225	limited	4	60/80	0.010	1.75	
Forsmark	Gneiss-granite	40	un-limited	13	50	0.027	0.14	1
Himmer-fjärden	Gneiss	30	limited	—	4.5–11	0.0005	—	2, 3
Gothenburg	Gneiss and granite	—	limited	—	12	0.0025	—	4
Gothenburg	Granite	—	limited	—	5	0.0017	—	4

1. A. Carlsson and T. Olsson (1977c), 2. A. Eriksson (1975), 3. Sund *et al.* (1977), 4. Palmqvist (1975).

Fig. 22. Distinct water inflow in an inclined zone of crushed rock within water-bearing zone D in the tailrace tunnel (Cf. Fig. 21). The capacity of the inflow is about 50 l/s. Photograph by Ö. Eriksson.

random fractures represent as much as 45 per cent of all measured fractures.

One comprehensive water-bearing zone was found in the headrace tunnel marked G. The capacity of this inflow initially amounted to about 70 l/s, but decreased to 25 l/s. The inflow emanates from a 25 m wide, steeply dipping tectonic zone.

Besides these 7 dominant water-bearing zones the residual inflow derives from individual fractures or minor zones of fractured or crushed rock in an otherwise tight rock mass. Hydraulically, the vertical joint set N60°E, seems to be the most important for groundwater leakage, and a number of minor inflows occur in fractures included in this set, because of their greater apertures, as observed in the tunnel-system and excavations in the machine station area. The classification system established for the water leakage in the Forsmark tunnel (A. Carlsson and T. Olsson 1977c:20) can be used to describe the groundwater leakage in individual fractures. The water leakage is thereby attributed to one of the five principles shown in Fig. 26. The characteristic features of the five types are as follows:

Table 11. The areas of large water inflow within the tunnel system and the capacities of the instantaneous and the stationary inflows in each area. The letters A–G are in accordance with the notations in Fig. 21.

Tunnels	Inflow area	Width m	Instantaneous inflow, l/s	Stationary inflow, l/s	Instantaneous inflow, l/s m	Stationary inflow, l/s m
Tailrace	A	225	15	5	0.07	0.02
Tailrace	B	250	60	30	0.24	0.12
Tailrace	C	35	85	35	2.4	1.00
Tailrace	D	20	180	35	9.0	1.75
Cable shaft	E	20	20	2	1.0	0.10
Access	F	100	100	50	1.0	0.50
Headrace	G	25	70	25	2.8	1.60

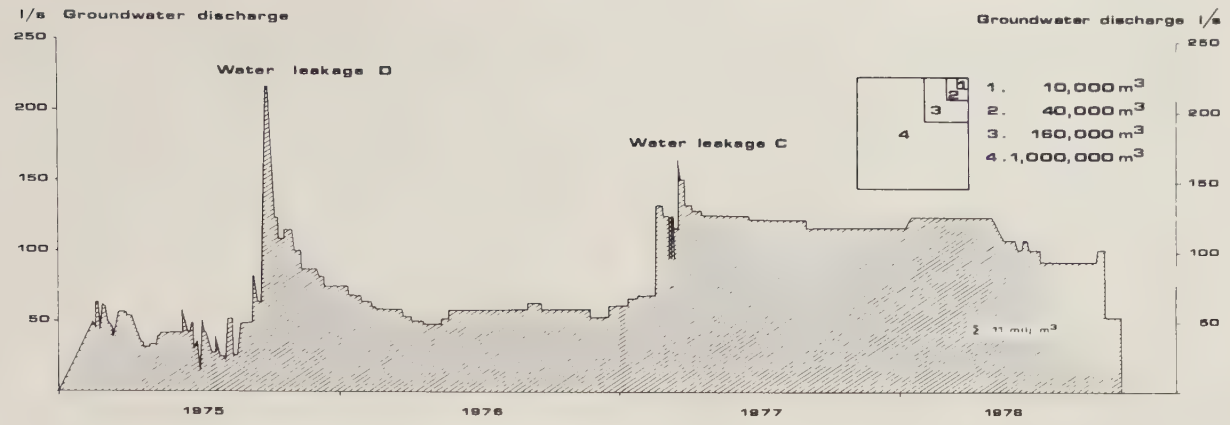


Fig. 23. Measurements of the water discharge at the measuring station in the intermediate access tunnel. The large water leakages C and D are clearly indicated.

1. The water inflow is concentrated to apertures in an otherwise tight fracture. In the Forsmark tunnel this condition applied mainly to horizontal fractures.
2. The water inflow follows horizontal or medium-steep fractures, often as a distinct inflow without noticeable apertures.
3. The water inflow occurs in steep fractures, generally as distinct inflows at the lower boundary of the fracture or on the tunnel floor.
4. The water inflow occurs at intersections of two or more fractures. Generally, no or only very small inflows occur along individual fracture planes outside the points of intersection.
5. The water inflow occurs as a diffuse inflow or discharge from rock on a broad front. In general, no distinctions between different inflows are noticeable and no correspondence is found between a certain water inflow and a certain fracture.

Whereas the water leakage in the Forsmark tunnel to a great extent occurred at horizontal fractures, these are less relevant to the inflows in the Juktan tunnels. The inflows in the Juktan tunnels are mainly attributed to types 1 and 4. However, type 1 leakages occur mainly in steeply dipping fractures. Within fracture zones and zones with crushed rock the type 4 is most frequent.

Groundwater inflow in different rock types. The rock type is essential for the development of water-bearing fractures. According to the disposition to brittle fracturing, different rock types produce joint systems with different hydraulic properties. Sund *et al.* (1977:54) pointed out that among the intrusive rock types, the granites usually have a more advanced, interconnected joint system, than basic intrusives. The joint pattern in metamorphic rocks, such as

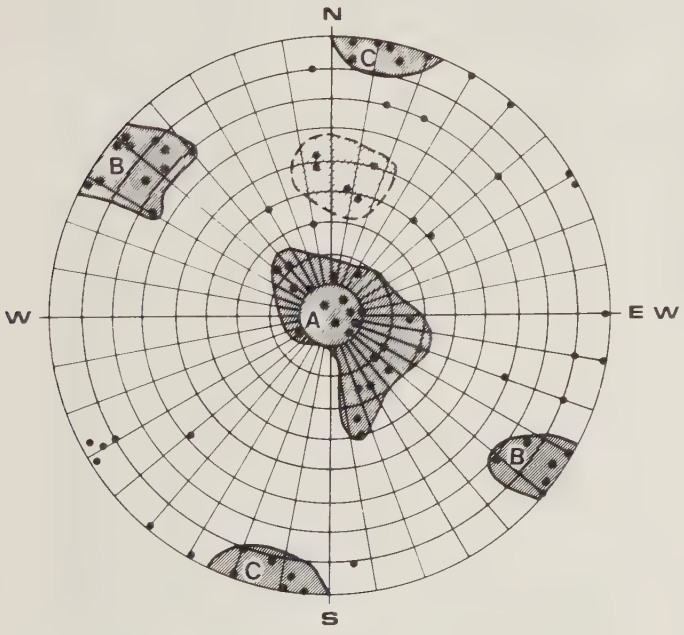


Fig. 24. Stereographic projection, showing the orientation of the fractures within zone F. The joint sets are marked by a special tone.

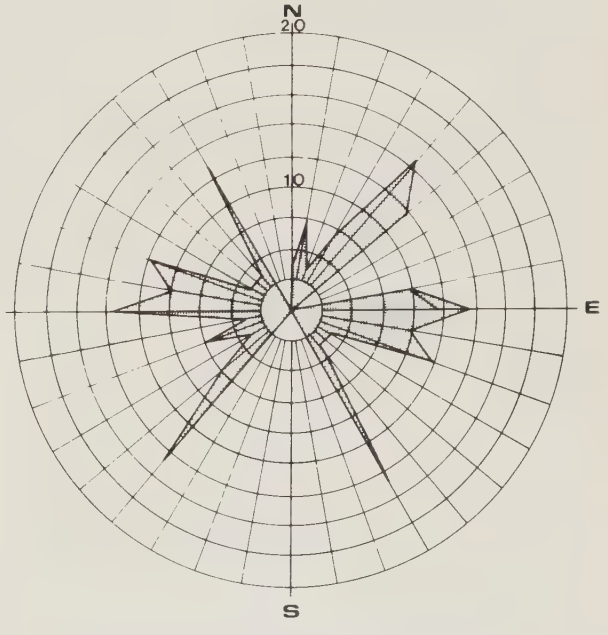


Fig. 25. Distribution of orientations of vertical and inclined fractures within zone F.

gneisses and schists, seems to vary greatly as regards interconnection and hydraulic conductivity. In granites the existing joint sets normally intersect each other, rendering the joint system hydraulically connected. On the other hand, fractures in gneisses and schists are normally incorporated in the schistose structure of these rocks. As a rule, this is the only well developed joint set to give a joint system with limited interconnections and consequently a rock mass with low hydraulic conductivity. These relationships between different rock types were also demonstrated by e.g. Le Grand (1949:117), Wenner (1951:3) and A. Carlsson and T. Olsson (1977b:305, 1977c:25).

Two main rock types occur in the Juktan area, namely a homogeneous, massive Revsund granite and a graphite bearing, schistose metasediment. The Revsund granite has a typical granitic joint pattern, consisting of a number of interconnected joint sets. In this case, four sets are present, the usual system being three. One of the sets is more or less horizontal while the others are vertical or subvertical, which creates a hydraulically interconnected joint system. One of the vertical sets includes a somewhat higher frequency of open fractures (N60°E), which, together with the horizontal joint set, seems to be crucial for the water-bearing capacity of the rock mass (A. Carlsson and T. Olsson 1978a:378).

As seen from the tectonic description of the area, the fracture frequency is generally much higher in the uppermost 20 m of the rock than at greater depth. Below the 20 m-level, the fracture frequency varies and no distinct relation versus depth is obtained (cf. Table 1).

This variation in fracture frequency or spacing versus depth is in agreement with conditions reported by A. Carlsson and T. Olsson (1980a) and A. Carlsson (1979) from a gneiss granite in the Forsmark area. A similar condition was also indicated by Snow (1968b:175), while Jahns (1943:73) is of the opinion that the fracture spacing and consequently, the fracture frequency changes rapidly with increasing depth. However, Johnson (1970:389) treated the results obtained by Jahn's mathematically and concluded that the fracture spacing increases rapidly down to a critical depth, after which the fracturing seems almost constant.

The difference in leakage in the main rock types in the Juktan area, with a higher inflow capacity in the Revsund granite—0.002 l/s m in the granite compared with 0.001 l/s m in the metasediment, excluding tectonic zones—is probably due to different fracture characteristics.

This relation between the main rock types agrees with the results presented by A. Carlsson and T. Olsson (1977c:25), who found twice as high an inflow capacity in a granitic rock as in a gneiss of sedimentary origin (0.038 l/s m v. 0.022 l/s m, including tectonic zones). According to Wenner (1951) the water-bearing system is usually better developed in granitic than in gneissic rocks.

Studies of the water leakage in the tunnel system showed that the occurrence of the groundwater differs in different parts of the rock mass and that these differences are chiefly due to a varying degree of fracturing. Heavily tectonized zones with a high frequency of open fractures contain large volumes of groundwater and permit fast transportation of water due to high storativity and high hydraulic conductivity. Consequently rock masses of intact rock with few fractures have low storativity and low conductivity.

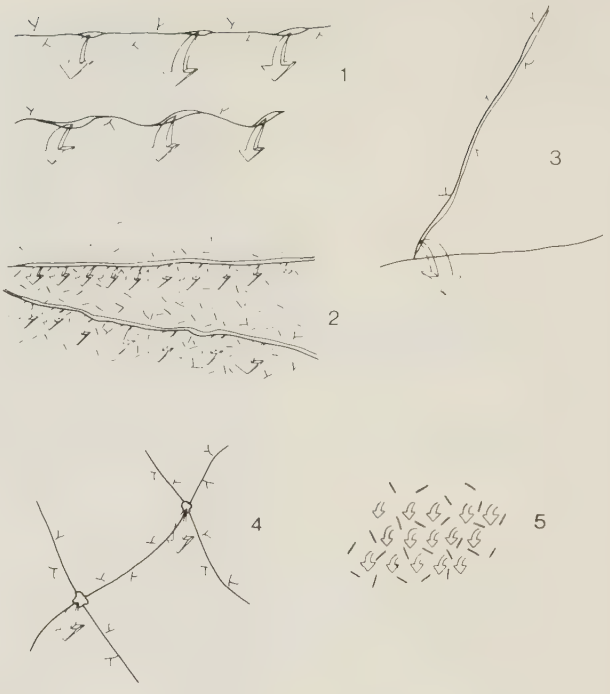


Fig. 26. Schematic illustration of the different appearances of the water inflows. 1. Aperture in a fracture. 2. The inflow along the exposed plane, mainly of horizontal or medium-steep fractures. 3. Inflow at the lower limit of a steep fracture. 4. Inflow at the intersections between different fractures. 5. Diffuse water inflow (A. Carlsson and T. Olsson 1977c).

Dating of the groundwater. During the construction of the plant, samples of the leaking water were analysed for tritium content. Furthermore, the Geological Survey performed tritum analyses and radiocarbon datings of the groundwater (Gidlund 1978). The Power Board carried out analyses on 31 water samples and the Geological survey accomplished 41 tritium analyses and 2 radiocarbon datings. The results of the radiocarbon datings of water from the headrace and tailrace tunnels at the intermediate access tunnel show ages of about 4800 BP and 9800 BP respectively (Gidlund 1978:16). The results of the tritium analyses carried out by the Power Board are presented in Table 12, and the changes in time for some sampling points in Fig. 27. Two series of surface water, one from Lake Blaiksjön and one from a tarn on Mt. Storblajken are included for reference. The majority of the groundwater analyses revealed somewhat lower tritium content than the surface water, but only two samples have a very low tritium content and these were collected in the outlet of the tailrace tunnel. It is obvious that leakage consists mainly of surface water, and that no distinct trends exist, which could indicate an increasing amount of surface water. However, only one series of groundwater analyses were continuously sampled, namely leakage C in the tailrace tunnel where the water showed a constant tritium content throughout 1977.

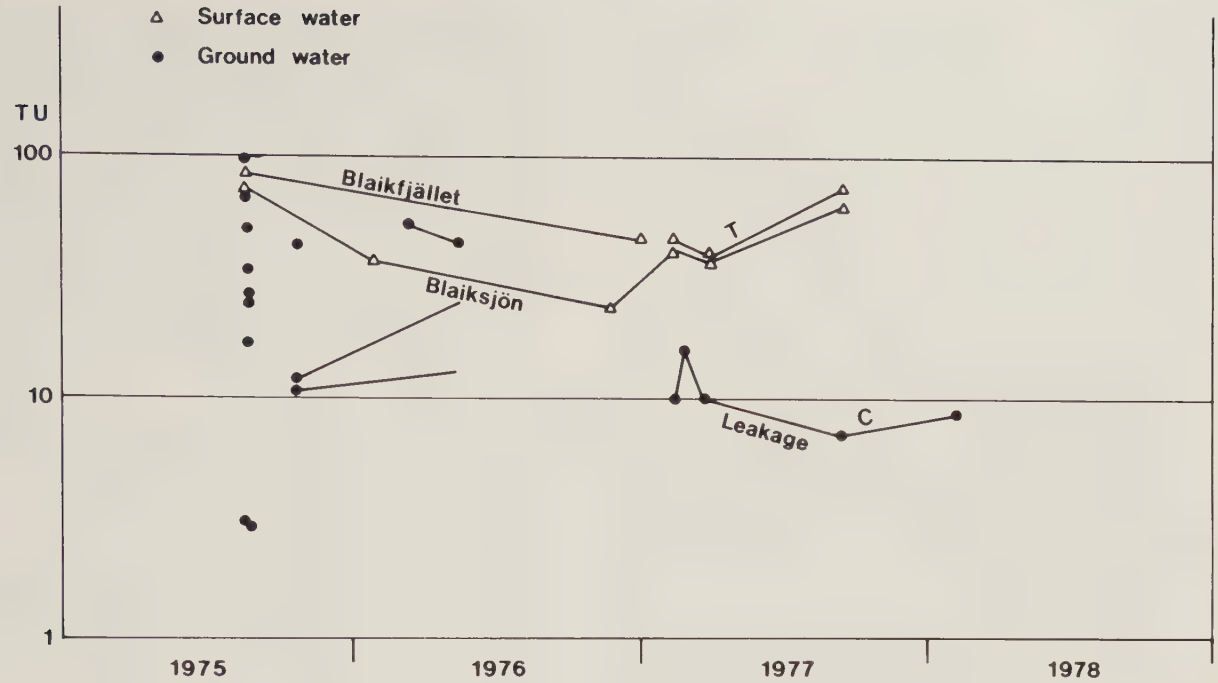


Fig. 27. Variations in tritium content (TU) in groundwater and surface water from the Juktan area. C. Leakage C. T. Tarn.

Hydraulic properties of the rock mass

Hydraulic conductivity determinations of the crystalline basement rocks in Sweden were performed by e.g. L. Carlsson and A. Carlstedt (1977), A. Carlsson and T. Olsson (1977a) and Hult *et al.* (1977). The conducitivity values obtained for the rock mass are generally varying. Normally, the conductivity ranges from about 10^{-8} m/s up to about 10^{-6} m/s, but the variations may be great even between neighbouring points. A. Carlsson and T. Olsson (1977a) found the conductivity of Revsund granite to be about 10^{-6} m/s. The conductivity of tectonic zones was studied by L. Carlsson and G. Ejdeling (1979), who executed numerical calculations of injection tests in tectonic zones in order to determine the hydraulic conductivity. In comparison the rock matrix generally has very low conductivity and the existing groundwater flow follows microfractures and joints between individual crystals. Investigations carried out by *inter alios* Heimli (1974), Lundgren (1978), and Rehbinder (1978), indicate that the conductivity of the rock matrix ranges from about 10^{-15} m/s to 10^{-10} m/s. The variations are due to the character

Table 12. Tritium content (in TU) in water from the sampling stations in the Juktan area.

Sampling station		Sampling month											
		75–08	75–09	75–10	76–01	76–03	76–05	76–11	76–12	77–02	77–03	77–09	78–02
Surface water													
Blaiksjön		75±2	–	–	37±4	–	–	24±2	–	41±2	38±3	62±3	–
Blaikfjället		85±2	–	–	–	–	–	–	45±2	45±2	39±2	75±3	–
Groundwater													
Headrace tunnel	*1*	69±5	–	–	–	–	–	–	–	–	–	–	–
Headrace tunnel	*2*	33±4	–	–	–	–	–	–	–	–	–	–	–
Access tunnel	*3*	90±2	–	–	–	–	–	–	–	–	–	–	–
Access tunnel	*4*	50±2	–	–	–	–	–	–	–	–	–	–	–
Access tunnel	*5*	17±2	–	–	–	–	–	–	–	–	–	–	–
Cable shaft	*6*	–	–	43±3	–	–	–	–	–	–	–	–	–
Machinery hall	*7*	–	–	12±3	–	–	25±3	–	–	–	–	–	–
Butterfly hall	*8*	–	–	–	–	52±3	44±4	–	–	–	–	–	–
Tailrace tunnel	*9*	–	–	11±3	–	–	13±3	–	–	–	–	–	–
Tailrace tunnel	*10*	27±3	–	–	–	–	–	–	–	–	–	–	–
Tailrace tunnel	*11*	24±3	–	–	–	–	–	–	–	–	–	–	–
Tailrace tunnel	*12*	3	–	–	–	–	–	–	–	–	–	–	–
Tailrace tunnel	*13*	3	–	–	–	–	–	–	–	–	–	–	–
Tailrace tunnel	*14*	–	–	–	–	–	–	–	–	10±2 16±2	10±3	7±4	9±3

and extent of microdiscontinuities. In Stockholm granite which resembles Revsund granite, Rehbinder (1978:25) found a fracture conductivity of 10^{-9} m/s, which corresponds to a rock mass conductivity of about 10^{-14} m/s. Heimli (1974:3) and Lundgren (1978:86) report values in the range 10^{-13}–10^{-10} m/s for different rock types.

The hydraulic conductivity of a formation is in proportion to the fourth power of the radius of the pores (Hagen-Poiseuill's law), which in a fractured formation corresponds to the third power of the aperture, and to the first power of the breadth of the fractures. These relations were further developed by *inter alios* Versluys (1915), Lomize (1951), Snow (1965) and Louis (1967). Versluys (1915:358) demonstrated the possibility of replacing a number of randomly oriented tubes by a set of three perpendicular tubes giving the same vectorial discharge. Snow (1965) adopted a similar approach to the problem of anisotropic permeability, but modelled the flow in parallel plates instead of tubes and applied tensorial notation in his derivations. This parallel plate model was further modified by Bianchi and Snow (1969).

Lomize (1951) and Louis (1967), after laboratory studies, introduced the roughness of the fracture walls as a fundamental parameter. The roughness induces the groundwater flow in a fracture to become non-parallel, so that the roughness is important for the hydraulic conductivity of a fracture.

Determinations of hydraulic conductivity

In order to quantify the variations in hydraulic conductivity in different parts of the rock mass in the Juktan area, two methods for determination were used, namely water-pressure tests in drill-holes and calculations based on the groundwater leakage in the tunnels. Determinations based on data from water wells were used as well but to a minor extent. These different methods of determination provided conductivity values from all parts of the investigated area, revealing a regional variation in conductivity. The validity of the obtained conductivity values was checked by analytical calculations of the groundwater balance and by numerical calculations performed in a small part of the area investigated. These calculations are presented below in "Groundwater balance".

Water-pressure tests. Since the introduction of the water-pressure test in 1933 (Lugeon 1933), the technique has been used throughout the world for engineering geological purposes. Yet the method is rarely used in the context of site exploration to discover the hydraulic conductivity of the rock mass but merely to classify the rock in terms of relative tightness and groutability.

The water-pressure test may be described as a reversed pumping test (pumping in) in which water under pressure is forced into the rock mass. Measurements of the water intake of the rock mass make it possible to determine the hydraulic conductivity of the aquifer. The field performance of the test and sources of error were discussed by Maini (1971), A. Carlsson and T. Olsson (1979a), Gidlund *et al.* (1979) and Wilson *et al.* (1979).

In all, 24 drill-holes were investigated with the water-pressure test, (Moberg 1973). In these measurements the water pressure was gradually increased to a maximum of about 500 kPa, after which the pressure is gradually relieved. Five load increments, 100, 300, 500, 300 and 100 kPa, were used as a rule. In each load increment, the pressure was constant for a certain period (normally 1—3 min) and the quantity of water consumed was measured. The equipment used for these measurements permits testing in rock masses with a conductivity down to about $5 \cdot 10^{-9}$ m/s (with a duration of test of 1 min and a test section of 3 m).

In the superficial rock mass, the tests were generally carried out as double packer tests (3 m interval between packers) and at greater depths as single packer tests. In the latter test the test section is limited by the packer and the bottom of the drill-hole.

Because of the measuring equipment used, it was impossible to perform the measurements during transience so stationary conditions were assumed to prevail. According to Moye (1967:99) who postulates that the rock mass is homogeneous and isotropic and the flow laminar, the following equation can be used to determine the hydraulic conductivity:

$$k = \frac{V}{2\pi HLt}\left(1 + \ln\frac{L}{d}\right) = \frac{Q}{2\pi HL}\left(1 + \ln\frac{L}{d}\right) \qquad (12)$$

where k = hydraulic conductivity of the rock mass (m/s)
H = net water head acting in the test section (m)
L = length of the test section (m)
V = volume of water consumed during the test (m^3)
t = duration of test (s)
Q = volume of water in unit time (m^3/s)
d = diameter of the test section (m)

L. Carlsson *et al.* (1979a) showed that the difference in results obtained is small between calculations according to transient or stationary flow conditions. In an aquifer with a conductivity of 10^{-8} m/s both assumptions give the same result.

Due to the short measuring times, transience usually prevails, wherefore the calculations should also be made according to this condition. However, transient measurements call for modified measuring equipment, and their results are of minor importance for site exploration measurements made for dam foundation and constructions.

According to A. Carlsson and T. Olsson (1979a:74) the chief errors in the water-pressure test are:

Leakage in the pipe system or at the packers

Deformation of the rock mass due to the hydraulic pressure

Stationary flow conditions do not prevail

Determinations of hydraulic conductivity in Swedish crystalline basement rocks based on water-pressure tests (A. Carlsson and T. Olsson 1976, 1977a,b and 1979a, Hult *et al.* 1978, Gidlund *et al.* 1978 and L. Carlsson *et al.* 1979) are generally evaluated as stationary tests. But transient water-pressure tests may probably be used to a large extent in the future, particularly for measurements in low conductive aquifers and for special purposes apart from ordinary site investigations.

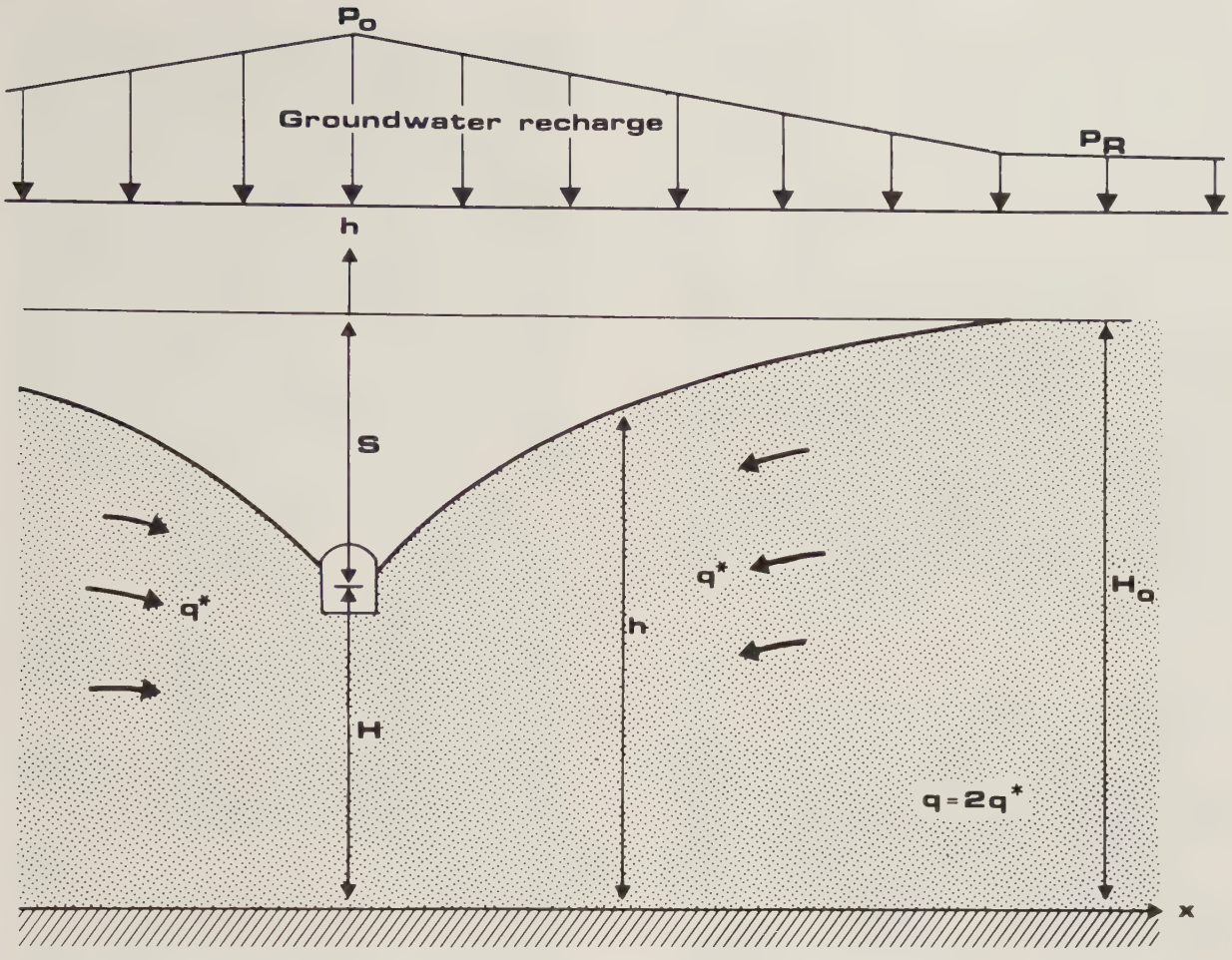


Fig. 28. Schematic profile through a rock mass drained by a tunnel. Principal notations in accordance with the notations used in the equations.

Conductivity determinations based on groundwater leakage. The amount of the groundwater which continuously leaks into a underground excavation is dependent on e.g. the hydraulic conductivity of the rock mass. Thus the water leakage is closely correlated with the conductivity which makes it possible to establish a mathematical relation based on the groundwater balance around the excavation which can be used to determine the hydraulic conductivity.

Examination of a section through a tunnel below the groundwater level, as shown in Fig. 28, confirms the following relations for the water balance. The existence of a tunnel causes a groundwater discharge according to Darcy's law:

$$q = -kh\frac{\partial h}{\partial x} \qquad (13)$$

where q = groundwater flow (m^3/s m)
k = hydraulic conductivity (m/s)
h = water bearing thickness of the formation (m)
$\partial h/\partial x$ = hydraulic gradient (m/m)

This discharge must be compensated by an equal recharge of water to the formation. Thus the following relation is valid if the recharge is due solely to infiltration of precipitation:

$$\partial q = P\ \partial x \qquad (14)$$

where P = groundwater recharge (m/s)

But because the groundwater level in the rock mass is lowered by drainage, a potential gradient between the rock mass and the soil cover is obtained which causes an increasing groundwater recharge, which is determined by the following factors:

Groundwater recharge to the soil aquifer
Vertical conductivity of the soil cover
Vertical conductivity of the rock mass
Difference in water head between the formations

The groundwater recharge to the soil cover is equal to the potential groundwater recharge to the rock mass, and is thus a maximum value. The vertical conductivity of the formations, together with the difference in head, determine the percolation rate.

Since the potential gradient, i.e. the difference in water head between the formations, depends on the distance from the tunnel, the term P in equation (14) is a function of the distance. The maximum groundwater recharge P_o occurs above the centre of the tunnel, and at some distance from the tunnel the recharge to the rock mass will be equal to the natural recharge, P_R (cf. Fig. 28). The assumption of

a linear decrease in recharge gives the following relation:

$$P = \left(\frac{P_R - P_0}{R}\right) x + P_0 \qquad (15)$$

where P = groundwater recharge at x m distance from the tunnel (m/s)
P_R = groundwater recharge to the rock mass under undisturbed conditions (m/s)
P_0 = induced groundwater recharge above the tunnel (m/s)
R = distance from the tunnel at which the recharge is P_R (m)

Combination of equations (14) and (15) and integration gives:

$$q = \left(\frac{P_R - P_0}{2R}\right) x^2 + P_0 x + C_1 \qquad (16)$$

The integration constant may be solved by using the boundary conditions given in Fig. 28.

$x = 0;\ q = q^*$
$x = R;\ q = 0$

which gives

$$C_1 = q^* \qquad (17)$$

and

$$q^* = -\frac{R}{2}(P_R + P_0) \qquad (18)$$

By combining and integrating equations (13) and (16) the following equation is obtained:

$$h^2 = C_2 - \left(\frac{P_R - P_0}{3Rk}\right)x^3 - \frac{P_0 x^2}{k} + \frac{R_x (P_R + P_0)}{k} \qquad (19)$$

The integration constant C_2 may be solved from the boundary conditions.

$x = 0;\ h = H$
$x = R;\ h = H_0$

and hence

$$k = \frac{R^2(2P_R + P_0)}{3(H_0^2 - H^2)} \qquad (20)$$

Substitution of R from (18) gives

$$k = \frac{4q^{*2}(2P_R + P_0)}{3(H_0^2 - H^2)(P_R + P_0)^2} \qquad (21)$$

which was the equation used for determining the hydraulic conductivity from data on the groundwater inflow in the tunnels. But the term P_0 must be defined before this equation can be used. This may be achieved by using an approximation of leaky aquifers. According to Huisman (1972:20) the following relation is valid:

$$P = k\frac{\Delta h}{H'} \qquad (22)$$

where P = groundwater recharge (m/s)
Δh = difference in head between the formations (m)
H′ = water-bearing thickness of the soil (m)
k′ = vertical hydraulic conductivity of the soil cover (m/s)

As the groundwater recharge to the soil aquifer represent the maximum recharge to the rock mass, it is possible to determine the value of P_0, i.e. the recharge above the tunnel axis. Generally, for fairly great k′ or Δh, $k'\Delta h > P_T$, which gives $P_0 = P_T$. This maximum recharge occurs above the tunnel axis. The recharge decreases laterally with increasing distance from the tunnel, and at the distance R the groundwater recharge will be P_R.

This value of the groundwater recharge during drainage due to tunnelling is approximately valid for the homogeneous rock mass. Drainage along conductive fracture zones causes an even higher recharge as the zones drain the surrounding rock mass (cf. A. Carlsson and T. Olsson 1978b:15). The amount of this increased recharge depends on the width of the fracture zone, in that the recharge into a narrow zone becomes relatively higher compared with a wider zone. By analogy with the derivation of (21), it is also possible to devise a general equation for the hydraulic conductivity based on water leakage along water-bearing zones. Using Darcy's law and the law of continuity, the following relation will be obtained after integration and determination of the integration constants:

$$k = \frac{4q^*(P_0 + 2P_R)B}{(H_0^2 - H^2)(P_R + P_0)^2(B + 0.67R_z)} \qquad (23)$$

where B = width of the water-bearing zone (m)
R_z = distance of influence over the surrounding intact rock mass (m)
q^* = half of the water inflow in unit length of the tunnel (m^3/s m)
P_0 = recharge above the tunnel axis (m/s)
P_R = recharge to the rock mass under undisturbed conditions (m/s)
H_0 and H are given in Fig. 28.

The derivation of this equation provides that the recharge above the tunnel is equal in the tectonic zone and the surrounding intact rock mass, which however is only possible if the recharge to the rock is governed by the recharge to the soil cover. Equation (23) is similar to equation (21) and the main difference is that in the latter the induced recharge is a function of the width of the water-bearing zone and the distance of influence into the surrounding rock mass.

In the determination of the hydraulic conductivity from the water leakage, equation (21) was used for the rock mass and equation (23) for fracture zones. The results are shown later in this section.

The dimension of the tunnels influences the quantity of leakage only insofar as the flow velocity will become very high near the tunnel if its dimension is small. Thus a turbulent flow may occur and the amount of the flow loss increase, which will reduce the amount of groundwater entering the tunnel. Consequently the calculation method underestimates the hydraulic conductivity when the tunnel dimension is small.

The above derivation presumes that an impervious layer intervenes at some level below the tunnel (cf. Fig. 28) and that the original groundwater level in the section is horizontal, i.e. no natural groundwater flow exists transverse to the tunnel. The first assumption is fulfilled in that the effect of variations in water-bearing height (H_0 in Fig. 28) is slight and the conductivity statistically decreases with increasing depth (A. Carlsson and T. Olsson 1977a:8, 1980b and below). The calculations postulate a theoretical impervious layer at the level at which the conductivity has decreased to about 20 per cent of that at the tunnel level. This gives a value of the water-bearing height (H_0) of 2.5 times the depth of the tunnel below the groundwater level.

Assuming that the groundwater level is parallel to the ground surface, only small gradients (<1 per cent) occur lateral to the tunnel axis. Thus the second presumption for derivation is approximately fulfilled. The great variations in gradient occur longitudinal to the tunnel, but do not influence the calculations.

Other papers in which the conductivity values based on groundwater leakage are reported usually apply other methods of calculation. M. Bergman (1977:232) presents an inventory of 73 underground caverns and tunnels in which he determined the hydraulic conductivity from the following assumptions:

The percolation area is equal to the surrounding rock surface, which in case of tunnels can be written $A=\pi dL$, where d is the equivalent diameter and L the length of the tunnel.

The acting hydraulic gradient is equal to 1.

The hydraulic conductivity in terms of inflow will thereby be:

$$k = \frac{q}{\pi d} \qquad (24)$$

where k = hydraulic conductivity (m/s)
q = groundwater inflow in unit length (m^3/s m)
d = diameter of the tunnel (m)

But this relation is a very rough approximation since the gradient is rarely governed by a constant head. Compared with the equations used for the calculations in the Juktan tunnels, this relation underestimates the hydraulic conductivity.

A. Carlsson and T. Olsson (1977c) and Lindblom (1979) report conductivity values from underground excavations which are situated and constructed so that the groundwater level above the excavations has a constant head. A. Carlsson and T. Olsson studied a submarine tunnel, the Forsmark tunnel, while Lindblom investigated a rock cavern with an infiltration gallery.

Water well data. Data from wells drilled in rock in the Juktan area were used to determine the hydraulic conductivity of the rock mass in which the wells are situated. These wells were treated with the aid of the regional parameters according to Jetel and Krásný (1968:460). L. Carlsson and A. Carlstedt (1977) modified the method to increase its application to Swedish conditions. The following relation is obtained from the capacity and depth of the well, drawdown and pumping duration.

$$k = \frac{Q}{a\mu ms} \qquad (25)$$

where Q = capacity
m = depth of the well
s = drawdown
$a\mu$ = factors depending on the well loss and pumping duration

According to L. Carlsson and A. Carlstedt (1977:106, 107) the equation (25) may be simplified as follows for ordinary wells drilled in crystalline rocks:

$$k = 1.25\frac{Q}{ms} \qquad (26)$$

This relation was used to determine the hydraulic conductivity from well data.

The method was applied in Sweden with plausible results by e.g. L. Carlsson and A. Carlstedt (1977) and A. Carlsson and T. Olsson (1976 and 1977a, b).

Result of hydraulic conductivity determinations. The hydraulic conductivity of the rock mass in the Juktan area was determined by means of water-pressure tests, water leakage determinations and well statistics. Mean values and variations for both the homogeneous rock mass and the penetrated fracture zones are shown in Table 13.

The hydraulic conductivity ranges through decades from below 10^{-9} m/s up to about 10^{-4} m/s. The conductivity of the fracture zones however, is fairly closely confined to the range 10^{-5}—10^{-4} m/s; the conductivity values of the homogeneous rock mass are more scattered but cover the range 10^{-8}—10^{-6} m/s.

Depth dependence and anisotropy of the hydraulic conductivity

Depth dependence. The reported values of the hydraulic conductivity of the rock mass were obtained at different depths below the rock surface. Water-pressure test defined the conductivity mainly in the superficial rock mass, while the determinations based on water leakage in some areas are valid for the rock mass at depth of several hundreds of meters. The conductivity varies widely chiefly beacause of different fracture characteristics and degree of ruptural tectonism. Furthermore, the conductivity decreases with increasing depth. This state of affairs was demonstrated by A. Carlsson and T. Olsson (1977a:83 and 1980b) as regards the Swedish crystalline basement rock types. The

Table 13. Hydraulic conductivity of the homogeneous rock mass and the fracture zones in the Juktan area.

Method of determination	Location	Hydraulic conductivity (m/s)
Zones of fractured or crushed rock		
Water-pressure test	Blaiksjön	$7\cdot10^{-6}$—$7\cdot10^{-5}$
Water leakage	Zone A	$2\cdot10^{-6}$
"	Zone B	$6\cdot10^{-5}$
"	Zone C	$1\cdot10^{-4}$
"	Zone D	$6\cdot10^{-5}$
"	Zone E	$1\cdot10^{-5}$
"	Zone F	$7\cdot10^{-5}$
"	Zone G	$3\cdot10^{-5}$
"	Mean value	$4\cdot10^{-5}$
Homogeneous rock mass		
Water-pressure test	Blaiksjön	$8\cdot10^{-9}$—$5\cdot10^{-7}$
Well statistics	Renberg	$3\cdot10^{-8}$—$6\cdot10^{-7}$
Water leakage	Tunnel system	$9\cdot10^{-9}$
Weighted mean value of the whole tunnel system		$1\cdot10^{-6}$

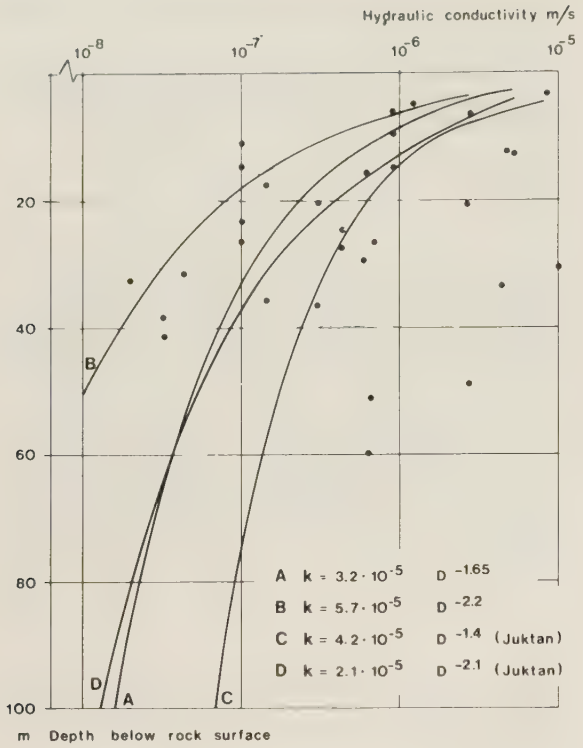


Fig. 29. Hydraulic conductivity versus depth.

obtained relations between the conductivity and the depth take the following forms:

$$k = 3.2 \cdot 10^{-5} \cdot D^{-1.65} \quad (27)$$

$$k = 5.7 \cdot 10^{-5} \cdot D^{-2.2} \quad (28)$$

where k = hydraulic conductivity (m/s)
D = depth below rock surface (m)

These equations are based on measurements from different areas chiefly in southern Sweden. The possible depth dependence of the conductivity in the Juktan area, was determined with the help of conductivity values from water-pressure tests.

Figure 29 shows the hydraulic conductivity versus depth from tests made in the vicinity of Lake Blaiksjön. The hydraulic conductivity decreases with increasing depth down to about 45 m below the rock surface. Below this level, down to about 60 m, the conductivity is again greater due to the existence of a flat tectonic zone (cf. leakage E). It was not possible to measure the rock mass beneath the 60 m-level because of poor rock quality at the 45—60 m-level, which prevented use of the instrumentation in the drill-holes at greater depths.

On the basis of all measurements carried out, the depth dependence of the conductivity at Juktan was determined by statistical methods. In this respect the same methods were used as in the analogous studies executed by A. Carlsson and T. Olsson (1977a,b and 1980b). The relation of the conductivity to depth was calculated by means of two different approaches; firstly, from all individual results and secondly, from the results obtained in homogeneous rock mass excluding strongly fractured zones. The obtained relations are as follows:

$$k = 2.1 \cdot 10^{-4} \cdot D^{-2.1} \text{ (homogenous rock mass)} \quad (29)$$

$$k = 4.2 \cdot 10^{-5} \cdot D^{-1.4} \text{ (rock mass including tectonic zones)} \quad (30)$$

The correlation coefficients are 0.65 and 0.36 respectively.

Thus the results of this treatment are in agreement with those presented by A. Carlsson and T. Olsson (1977a,b and 1980b). The main difference is that the rock mass in the Juktan area appears more conductive. Theoretically, the hydraulic conductivity at 10 m depth in the Juktan area is about $1.7 \cdot 10^{-6}$ m/s which satisfies both equations (29) and (30). By analogy, the conductivity at 10 m depth is $7.2 \cdot 10^{-7}$ m/s and $3.6 \cdot 10^{-7}$ m/s based on equations (27) and (28). The gradients (exponents) in equation (28) and (29) are alike, and both equations represent rock masses with few fractures. Equation (29) reports the trend in the superficial part, down to 45 m, in the Juktan area and equation (28) illustrates a similar circumstance at Forsmark. The same conditions also hold for equations (27) and (30) which report the trends in integrated rock masses including both fracture zones and the homogeneous rock mass. The depth dependence therefore seems to be more distinct in the homogeneous than in densely fractured rock, as seen from the exponents.

If these equations are also valid at greater depths, the conductivity can be determined at the level at which the machine hall is located, i.e. 250 m below the rock surface. According to equations (29) and (30) the conductivity at this depth would be about $2 \cdot 10^{-9}$ m/s in the homogeneous rock mass and about $2 \cdot 10^{-8}$ for the rock in general, including both sparsely and heavily fractured zones. But this latter value is far too low compared with the average conductivity values in Table 13. This suggests that the depth dependence in fracture zones is lower than the values obtained.

Hydraulic anisotropy of the rock mass. Measurements were performed by the Power Board in the machine station area in order to evaluate the static water head and its influence on rock stability. For this purpose 38 drill holes about 10 m deep were drilled through the roofs of the excavations. The drill-holes were fitted with packers and connected to manometers with drain cocks. This instrumentation allowed monitoring of the water head.

The arrangement with manometers and drain cocks also provided for systematic measurements of the changes in head in different drill-holes due to a controlled disturbance at some point. In order to cause a disturbance in the groundwater system, the drain cock in one of the drill-holes was opened, reducing the head at that point. The effect on the water head in the other drill-holes was observed. Assuming that stationary flow prevails, the water velocity between two points will be constant and the induced gradient in proportion to the inversion of the hydraulic conductivity. The validity of this assumption was demonstrated by A. Carlsson and T. Olsson (1978a:385). By means of this relation it was possible to determine the variation in conductivity in different directions, via the obtained differences in head between the drill-holes. A measurement is shown in Fig. 30, in which drill-hole 9 was opened, whereupon a resulting decrease in head was observed in drill-holes 2 and 13. A similar decrease in

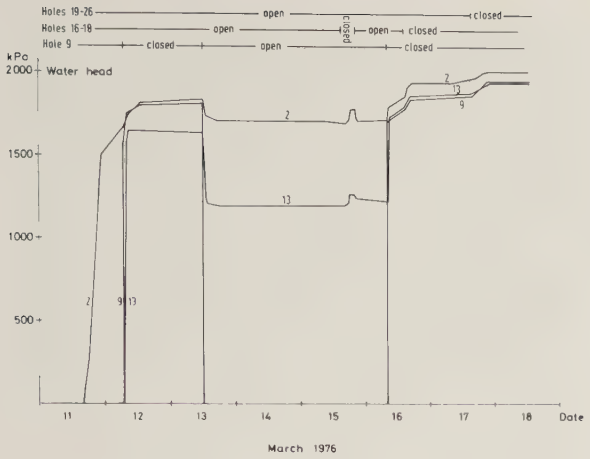


Fig. 30. Measurements of the water head in the rock mass at the machine hall area (A. Carlsson and T. Olsson 1978a).

head was also obtained in the other drill-holes, but this is not shown in the figure. As appears in Fig. 30, an approximately stationary state of flow arises fairly soon after the opening of drill-hole no. 9.

The water-head measurements reveal a marked anisotropy in the ratio of 3:1 between the two main directions, k_1 and k_2. The geographical orientation of the relation is given in Fig. 31. As the pseudo-isotropic hydraulic conductivity (determined by water-pressure tests and extrapolated to the present depth) is about $2 \cdot 10^{-9}$ m/s, the anisotropic conductivity can be written:

$$k_1 = 1.9 \cdot 10^{-9} \text{ m/s}$$

$$k_2 = 0.6 \cdot 10^{-9} \text{ m/s}$$

The hydraulic anisotropy in fractured aquifers is chiefly a result of different properties of the intersecting fractures, which is discussed by *inter alios* Warren and Root (1963), Snow (1969), Louis (1974a and 1974b) and Rocha and Franciss (1977).

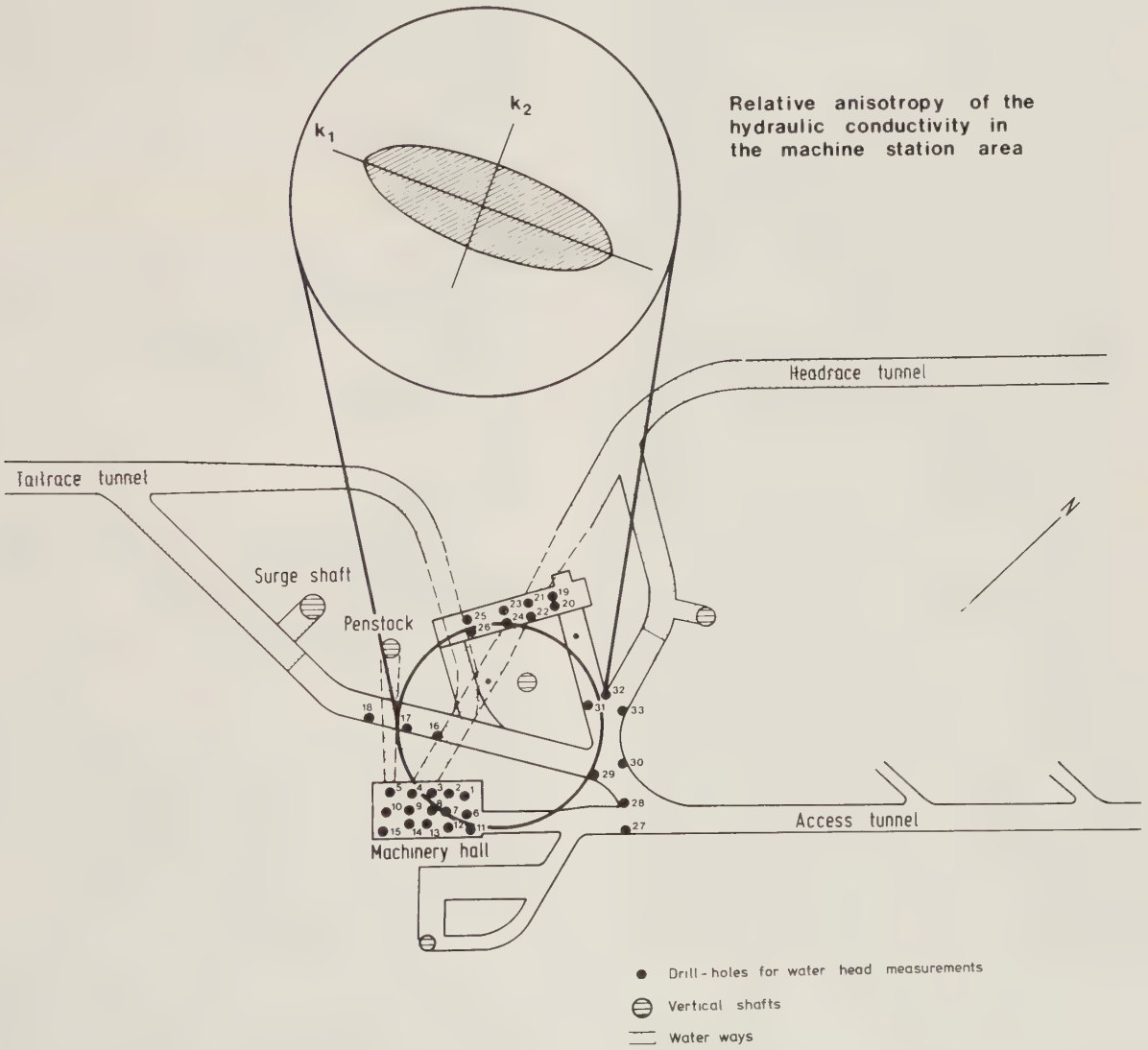


Fig. 31. Orientation of the anisotropy in the horizontal plane of the hydraulic conductivity. The drillholes in which the water-head measurements were made are marked.

Hydraulic conductivity and its relation to the rock stress distribution

The primary stresses acting in a rock mass derive from diverse origins. They may be subdivided into gravitational, tectonical and geological stresses (C. Jaeger 1972:82). The last two may be either active or paleoresidual in character. These stresses create a specific distribution of the stress field within the rock mass. According to S.G. Bergman (1977:2), the discontinuities present in a rock mass cause local variation in both the magnitude and the direction of the principal axis of the primary stress field. In this context, the topography is an additional factor of importance for the distribution of the primary stresses.

At almost every site where rock stress measurements have been performed, the results tend to show higher horizontal than vertical primary stresses. This condition is significant for at least the superficial rock mass (Hoeck and Brown 1977 and Haimson *et al.* 1977). As a rule the vertical primary stress corresponds fairly closely to the weight of the overburden as pointed out by e.g. Haimson *et al.* (1977:278). According to Stephansson and Leijon (1979:9) the high horizontal primary stress probably depends on unloading due to erosion and deglaciation.

Apart from this difference between the vertical and the horizontal primary stresses, the horizontal stress tensor normally follows different principal stresses (cf. Haimson *et al.* 1977:278 and Hiltscher *et al.* 1979:231). This circumstance with different principal stresses in different directions may be significant for the hydraulic conductivity of the rock mass, which indeed depends on the existing fractures. According to Walsh (1965:386) the fractures are in general more compressible than the intact rock matrix so that differences in effective stress on the fracture planes may change the conductivity.

Shear stresses however, will always beset the fractures which has an additional effect on the apertures. The relation between shear deformation and hydraulic conductivity is complex owing to the change in geometric configuration of sheared fracture. According to Sharp and Maini (1972:12) the shearing of hard rocks in general results in an increase in conductivity via dilation of the fractures at low normal stresses as shown in Fig. 32. However as also obvious in the figure, and as pointed out by Barton (1971:9), the shear displacement gives rise to a marked geometric anisotropy in the fracture void space. Parallel to the direction of shearing the fracture opening will be influenced by contacts between the fracture faces. The effect of the shearing depends, according to Barton and Choubey (1977:4), on the roughness of the fracture walls, the shear strength of the fracture walls and the normal stress.

The discussion indicates that the aperture of a fracture, and thereby the hydraulic conductivity, is dependent on the existing stress field. In an otherwise isotropic rock mass with identical fracture properties of different joint sets, the fracture apertures and the conductivity differ in different directions due to differences in the stress field. A joint set parallel to the greatest principal stress will have wider fractures and higher conductivity. This relation is schematically shown in Fig. 33.

The relation between hydraulic conductivity and effective stress or pore pressure was discussed by several authors, but the relation was usually attributed to induced stresses on rock samples and not to the existing primary stress field in the rock mass (Serafim and del Campo 1965, Snow 1968c, Brace 1978, Zoback and Byerlee 1975 among others). Witherspoon *et al.* (1979), who studied the size-effect on the hydraulic properties of fractures, found that the effect of applied stresses was higher for small than for large specimens. The conclusions suggest that the conductivity deduced from small rock cores may be significantly lower than would be found in the field under the same stress conditions.

The effect of the stress on the conductivity depends on whether the stress field closes or tightens the fractures. Pratt *et al.* (1974:43) showed by *in situ* tests that the permeability decreased by a factor of 4 when normal stress was applied across the fracture. But normal stress parallel to the fracture increased the permeability by a factor of 2.5.

Investigations of petroleum technology show that the conductivity of fractures is stress dependent, as documented by e.g. Fatt and Davis (1952:329), Gray *et al.* (1963:100) and McLetchie *et al.* (1958:388). Cooke (1973:1002) and Holditch and Morse (1976:1177) maintain that high closing pressure can reduce the fracture conductivity near a well. The reduction in conductivity is due to the decrease in hydraulic pressure during pumping to the well. In this case the effective stress increases due to the decreasing hydraulic head. This condition is also shown by A. Carlsson and T. Olsson (1979a:73) in connection with water-pressure tests in fractured rock masses using high testing pressures.

Lindblom (1973) presents calculations based on a

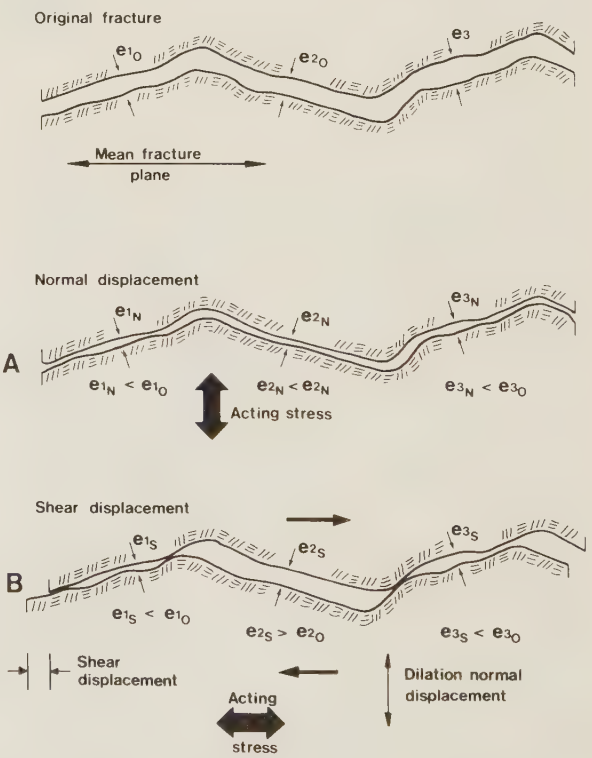


Fig. 32. Effects of the direction of the fracture plane within the stress field. A. Maximum principal stress perpendicular to the fracture plane. Effective change in fracture opening due to normal displacements. B. Maximum principal stress parallel to the mean fracture plane. Effective change in fracture opening due to shear displacements.

coupled flow-stress analysis using the finite element method. The method used was developed at the University of California, Berkeley and is described by Brekke *et al.* (1972).

The analysis carried out by Lindblom is applied to a practical tunnel problem. In a fractured rock mass with two water-bearing joint sets possessing identical fracture characteristics, a tunnel is situated 200 m below the original groundwater level. The original stress field consists of an effective vertical stress equal to the weight of the overlying rock mass, and a horizontal stress with a magnitude of half the vertical stress. Due to water leakage into the tunnel, the hydraulic head in the rock mass becomes lower and the effective stresses consequently become higher. The obtained changes of the stress field caused significantly different fracture apertures, as seen in Fig. 34. Some fracture openings were closed while others were widened.

The effect on the conductivity of changes in the stress field due to tunneling was also noticed by Martna (1970, 1971:144 and pers. com.). This study of the rock-bursting zone of the Sourva-Vietas headrace tunnel revealed that the water inflow started a few days after the excavations of the top-heading and were almost exclusively confined to fractures parallel to the maximal stresses.

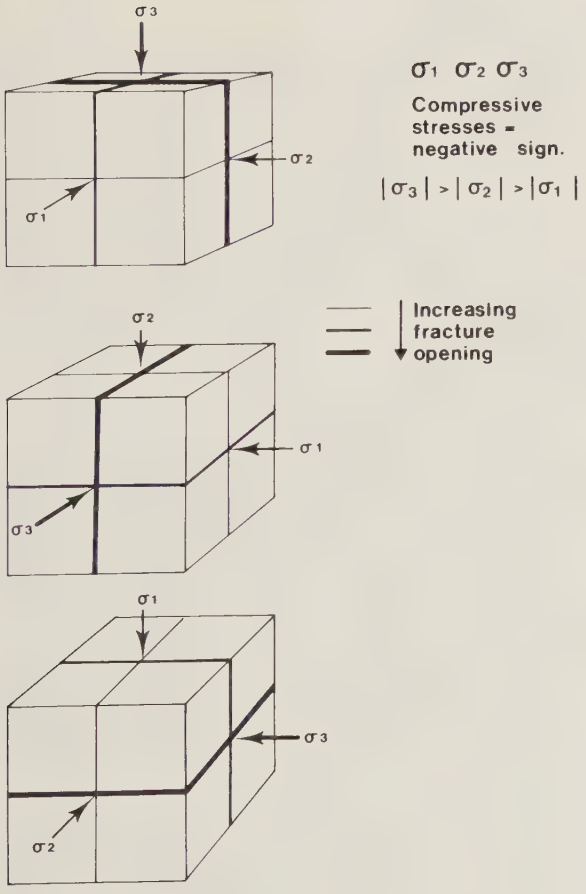


Fig. 33. Schematic illustration of the relation between the magnitudes of the principal stresses and the water-bearing ability of the different fractures.

Rock stress measurements in the Juktan area

Rock stress measurements were made at three different points in the machine hall area (Hiltscher 1975). The results show wide variations both in directions and magnitudes of the principal stresses, as seen in Fig. 35. The horizontal principal stresses average 10 MPa and 4 MPa respectively, while the vertical, corresponds fairly closely to the weight of the overburden, i.e. about 6 MPa. The measuring points are situated in the access tunnel to the machine hall, the access tunnel to the butterfly valve hall and in the penstock. Furthermore two measurements were taken outside the machine hall area in the headrace and tailrace tunnels, but these will not be discussed here.

For the measurements the well-known method of Leeman and Hayes (1966) was chosen. This method was further developed at the Swedish State Power Board by Hiltscher (1976) and the procedure is as follows. Three strain-gauge rosettes are cemented in a drill-hole and the strain conditions recorded, whereupon the stresses are released by overcoring. After the overcoring the strain conditions are recorded once more. The actual stress field is calculated from the strains recorded before and after overcoring. To obtain a reasonable mean value, several measurements were carried out in the same drill-hole close to each other.

The measurements are made in drill-holes piercing in the roofs of the different excavations at depths at which no stresses induced by the excavations are to be found which could influence the recordings. In view of the fact that the machine hall area has a very complex design with excavations on several floors and side by side, it is very difficult to avoid disturbances from the surrounding induced stress field.

This may explain the differences obtained between the measuring points. But as the anisotropy of the hydraulic conductivity was also obtained in connection with the excavations, the anisotropy of this parameter too is probably influenced by the induced stress field.

Considering the location of the rock stress measurements in relation to the water head measurements, it is feasible that the measuring point in the penstock would be the most representative.

The hydraulic conductivity in relation to depth and stress field

A comparison of the directions of the anisotropy of the hydraulic conductivity with the state of stress in the rock mass (Fig. 36), demonstrates a correspondence, as regards the directions, between the extreme values of the conductivity and the compressive stresses measured within the machine hall area. Almost perpendicular thereto, there is a slight tendency to tensile stress at some measuring points. Therefore it is reasonable to assume that fractures with a strike parallel to the highest compressive stress are kept open by the low compressive stress on the fracture planes and accordingly, the fractures almost perpendicular to the highest compressive stress are more closed. Thus the anisotropic behaviour of the hydraulic conducitivty seems to be stress dependent (A. Carlsson and T. Olsson 1979c).

It is also of interest to discuss whether or not the depth dependence of the conductivity is related to the stress field. Rock stress measurements at different depths below the

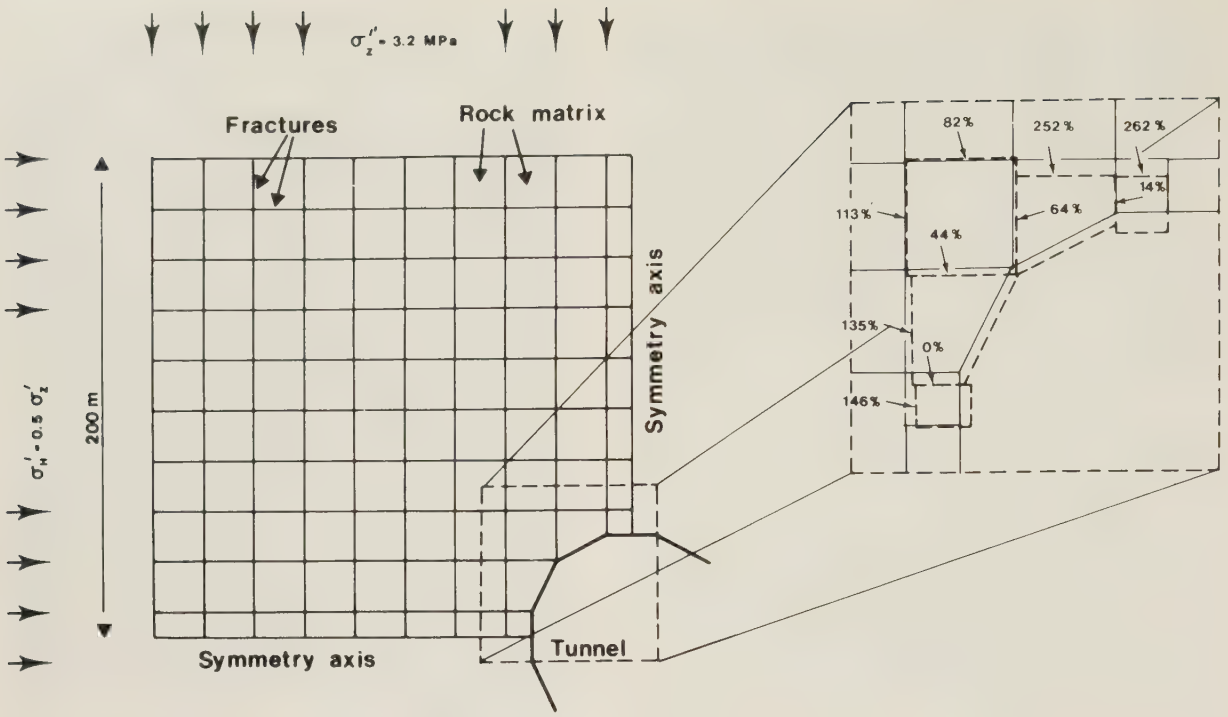


Fig. 34. Changes in fracture apertures due to changes in hydraulic head. The fracture apertures are given in per cent of the original apertures. Modified after Lindblom (1973).

rock surface would here prove useful. But no such measurements were made at Juktan, wherefore the following discussion is based on rock mechanical theories and measurements from other areas.

Hast (1958:173 and 1974:471) found that the horizontal stresses are aggravated by increasing depth and in general are much higher than the dead weight of the overlying strata. This however is not verified by other measurements (Hiltscher 1974, S. Bergman 1977 and Hiltscher *et al.* 1979). The increase in rock stress with increasing depth is on the whole much less in these works than in those of Hast. Most of the existing rock stress measurements derive from mines or underground excavations. Few measurements are so far available from drill-holes at different levels, and all were made in the construction area of Forsmark power plant (Hiltscher and Strindell 1976 and Hiltscher *et al.* 1979). These measurements which were usually taken every 20 m in one angular and three vertical drill-holes indicated very high horizontal rock stresses, but a minor increase with depth. The principal stresses at different depths in one of the drill-holes are shown in Fig. 37. According to Hiltscher *et al.* (1979:23) the increase in vertical stress is approximately in accordance with the weight of the overburden, although tensile stresses exist even at great depths. This condition, with the compressive stresses generally increasing with increasing depth, indicates that the fracture aperture decreases with increasing depth. Consequently the hydraulic conductivity also decreases with increasing depth.

The depth dependence of the hydraulic conductivity is weaker in tectonic zones than in the homogenous rock mass (cf. equations 27–30), probably because of a stress releasing effect of the fractured zones. Krantz *et al.* (1979:234), however, point out that the decrease in permeability is greater for fractures than for the intact rock. In addition, since fracture roughness affects the decline of permeability with pressure, some fractures will probably be active fluid conductors even at very high pressures. However, zones dense in fractures probably dilate the rock mass, and transmit great stresses with a minor deformation of each fracture, but the deformation within the zone as a whole is severe. This effect of fractures and fracture zones is verified by the different moduli of elasticity obtained for fractured and unfractured rocks (C. Jaeger 1972:85 and A. Carlsson 1979). Investigations at the Messaure hydro-power plant in connection with the construction of a 100 m high rock-filled dam (Bernell and Scherman 1979) indicate a clear relation between hydraulic conductivity and deformation of the rock mass. During the unloading due to excavation and the load imposed by the foundation of the dam, both the hydraulic conductivity and the subsidence of the rock mass were considerably changed.

These conclusions imply that both the hydraulic anisotropy and the depth dependence of the conductivity are closely correlated with the stress distribution in the rock mass. Naturally, this dependence is only definite if the fracture characteristics in different directions are about the same. Variations in foliation and fracture characteristics are probably more important for hydraulic anisotropy than the state of stress. In schistose or gneissic rocks the anisotropy mainly depends on the foliation.

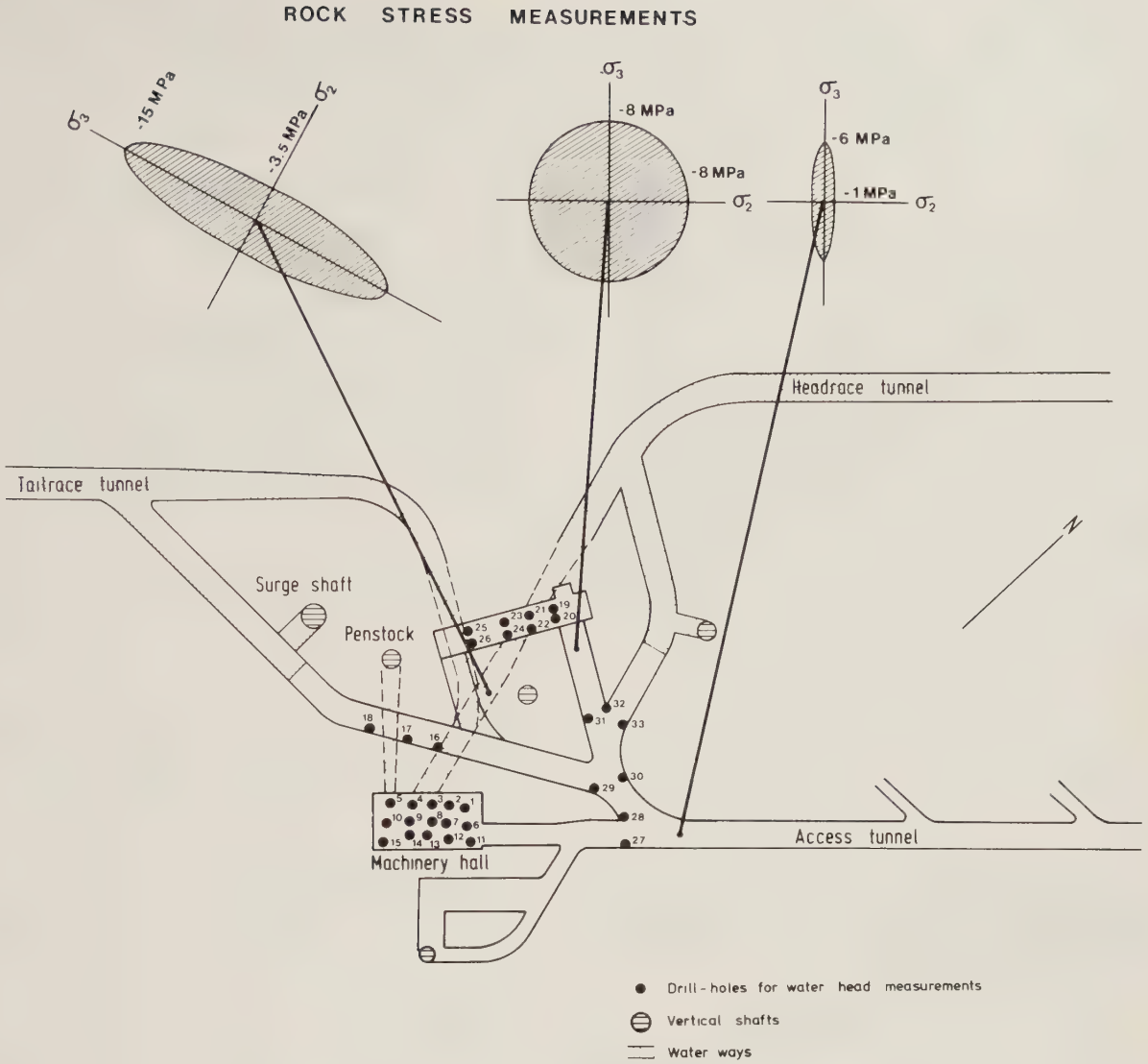


Fig. 35. Results from the rock stress measurements made in the machine hall area.

Fracture characteristics and kinematic porosity

Relation between the hydraulic conductivity and the fracture characteristics

In order to evaluate the quality of the individual fractures intersecting the rock mass in the Juktan area, some characteristic properties of the fractures were determined. Assuming that the fracture frequency or spacing is known, it is possible to calculate the fracture aperture and fracture conductivity. In addition, the kinematic porosity of the rock mass can be estimated.

In a rock mass consisting of rock material and transecting discontinuities, the hydraulic conductivity will constitute an integrated mean value of the conductivity of the different elements. The conductivity of a section with the length L and unit breadth will be:

$$k = \frac{1}{L}(b_1k_1 + b_2k_2 + \ldots\ldots\ldots + b_nk_n) \qquad (31)$$

where k = hydraulic conductivity of the section (m/s)
L = length of the section (m)
b_i and k_i = width and hydraulic conductivity of the single elements (m)(m/s)

This relation is illustrated in Fig. 38.

Assuming that all fractures included in the section have the same apertures and spacing, equation (31) becomes:

$$k = \frac{1}{b}(ek_j + (b-e)k_r) \qquad (32)$$

where k = hydraulic conductivity of the rock mass (m/s)
k_j = hydraulic conductivity of the fractures (m/s)
k_r = hydraulic conductivity of the rock material (m/s)
b = fracture spacing (m)
e = fracture aperture (m)

As seen from the conductivity values for the rock mass and the rock matrix reported above, the hydraulic conduc-

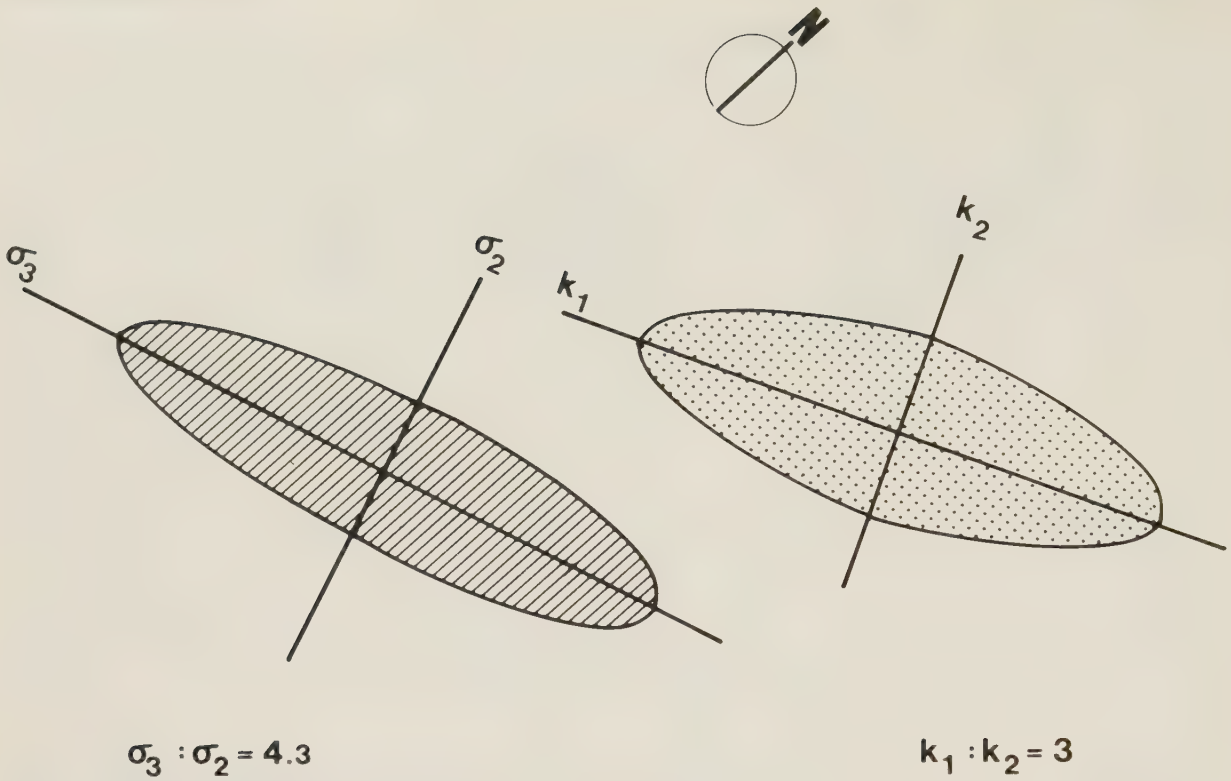


Fig. 36. Comparison of the relative magnitudes and the geographical orientation of the principal stresses and the anisotropic hydraulic conductivity.

tivity of the rock material in a crystalline rock mass is much less than that of the fractures, i.e. $k_r \ll k_j$, and the fracture apertures are smaller than the spacing ($e<b$). Hence, it is possible to reduce equation (32) to (cf. A. Carlsson and T. Olsson 1980b):

$$k = \frac{e}{b} k_j \tag{33}$$

which is equal to

$$k = nek_j \tag{34}$$

where n = fracture frequency (l/m)

Thus, the hydraulic conductivity of the rock mass is a function of fracture spacing/frequency, fracture aperture and fracture conductivity.

As mentioned above, Snow (1965) and Louis (1967) carried out laboratory studies of the relation between the hydraulic conductivity and fracture characteristics. Assuming parallel flow within smooth fractures Snow (1965) states the following relation.

$$k = \frac{ge^3}{12\upsilon b} \tag{35}$$

where k = hydraulic conductivity of the rock mass (m/s)
g = acceleration due to gravity (m/s^2)
e = fracture aperture (m)
υ = kinematic viscosity (m^2/s)
b = fracture spacing (m)

Considering the roughness of the fracture walls which divert the flow, Louis (1967:67) deduced that:

$$k = \frac{ge^3}{12\upsilon Cb} \tag{36}$$

where $C = (1 + 8.8(k_h/D_h)^{1.5})$

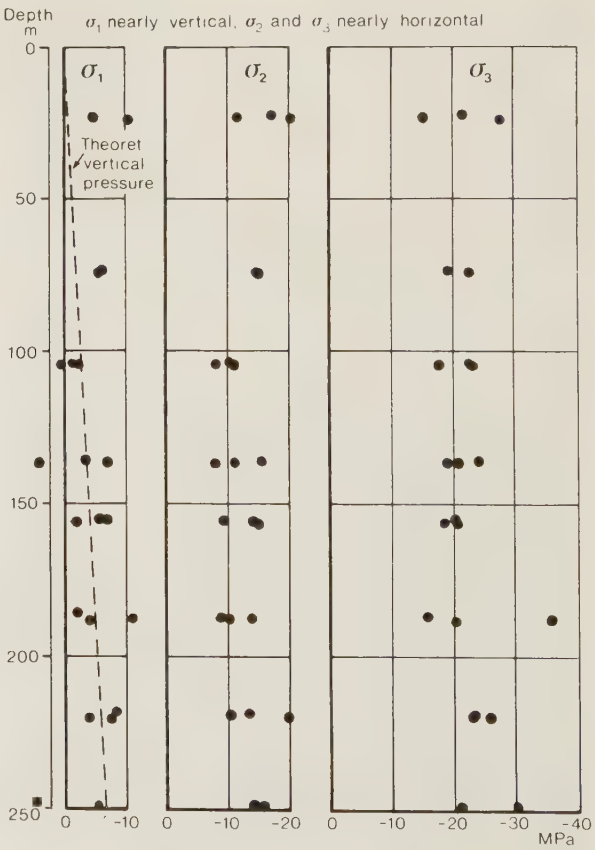


Fig. 37. Principal stresses versus depth obtained in drill-hole DBT-3 in the Forsmark area (Hiltscher *et al.* 1979).

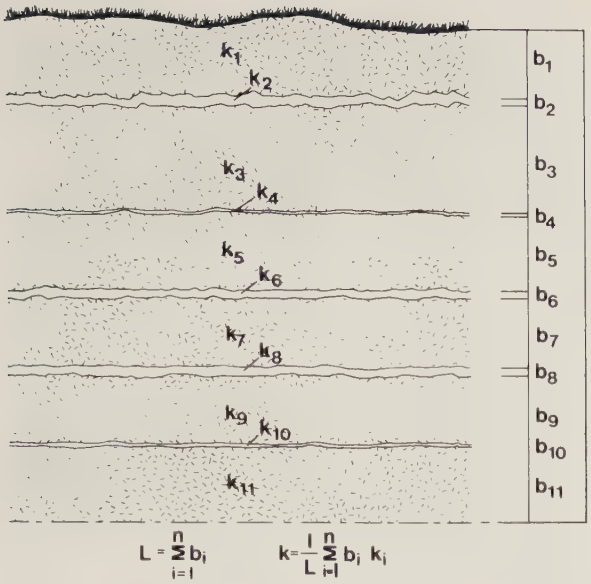


Fig. 38. Schematic view of the structural elements in a rock mass and their implications for the rock mass conductivity.

The function C describes the effect of the relative fracture roughness on the conductivity, the function being derived from laboratory experiments. According to Louis (1967:64) the term (k_h/D_h) is probably of the magnitude 0.4–0.5 in natural fractures, which gives a value for the C-function of about 4. This means that the conductivity in a rough fracture is about 25 per cent of the conductivity in a smooth one.

Groundwater flow in natural fractures normally takes place under non-parallel conditions, which indicates that equation (36) best satisfies natural conditions. Solving this equation with respect to the fracture aperture, *e*, gives:

$$e = \left(\frac{12k\upsilon Cb}{g}\right)^{1/3} \qquad (37)$$

which was used to determine the apertures of the existing fractures from the conductivity values and fracture frequency. This relation is graphically shown in Fig. 39. The method of determining the fracture apertures from the hydraulic conductivity was used by *inter alios* Bianchi and Snow (1969:238), and A. Carlsson and T. Olsson (1978c:129).

Nevertheless, the equation gives the apertures of a number of fractures situated within a single test section. A

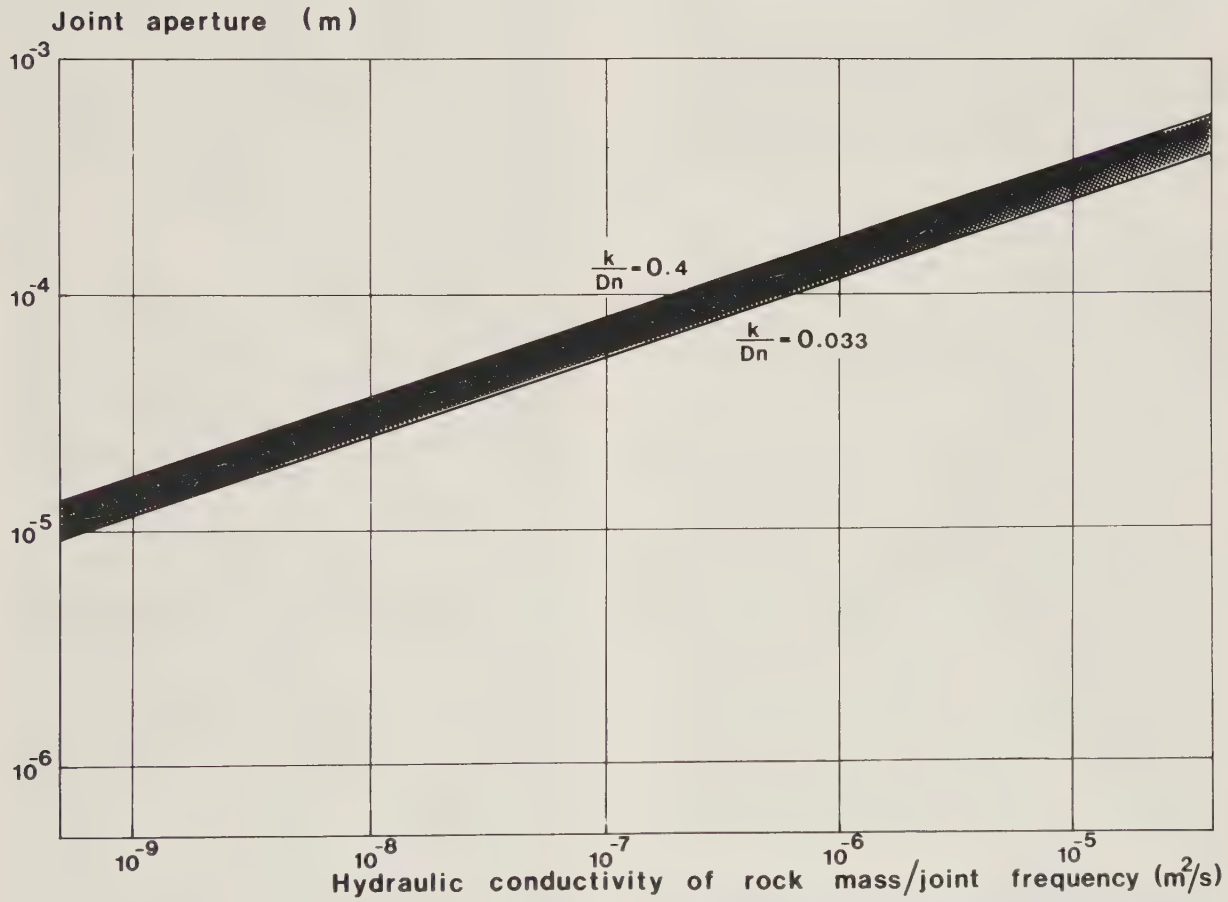


Fig. 39. The relation between the hydraulic conductivity of the rock mass, the fracture frequency and the fracture aperture for different values of the relative roughness of the fracture planes. The relative roughness is given by the term (k/Dn), where the values 0.4 and 0.033 represent rough and smooth fractures respectively (A. Carlsson and T. Olsson 1978c).

more accurate value of the fracture apertures calls for individual tests on each fracture, and the validity of the calculated aperture decreases with increasing size of the test section. Of the methods used in the Juktan area for determining the hydraulic conductivity, the water-pressure test is the only method to allow evaluation of the variations in apertures. In water-pressure tests the measurements are made in a test section of constant length in which only a few fractures may occur. Thus, the obtained fracture aperture is a mean value of all the fractures within the section. Furthermore, it is not feasible to consider the variation in aperture because irregularities in the fracture planes within a single fracture. The aperture calculated by equation (37) is equal to the essential aperture of an fictive fracture with a continuous, equal opening which can replace the natural fracture without changing the conductivity of the test section as a whole.

Conductivity values based on water inflow or single packer tests, represent parts of the rock mass, which are several orders of magnitude greater than the test section used in double packer tests. Compared with these measurements the values obtained by double packer tests must be more accurate since only a few fractures intersect the test section. With a known value of the fracture aperture it is possible to determine the fracture conductivity as seen in equation (34). Solving this equation with respect to the fracture conductivity gives:

$$k_j = \frac{k}{en} \tag{38}$$

which was used to illustrate the variation between different parts of the rock mass in the Juktan area. In that the aperture is a fictive mean value, the fracture conductivity too will be fictive. Nevertheless, this value is probably more accurate since the conductivity of both the rock mass and the fractures are governed by the narrowest sections of the fractures.

Kinematic porosity

According to Norton and Knapp (1977:915), the total porosity in a fractured medium, Θ_T, may be represented by:

$$\Theta_T = \Theta_K + \Theta_D + \Theta_R \tag{39}$$

where Θ_K = effective flow porosity or kinematic porosity
Θ_D = diffusion porosity
Θ_R = residual porosity

The kinematic porosity represents the fractures through which the dominant fluid flow proceeds, while the diffusion and residual porosity represent fractures in which no or only very limited flow takes place. The fractures making up the residual porosity are not connected with the fractures included in either the kinematic or the diffusion porosity. Figure 40 represents the geometric relationships between the different porosities.

In a fractured rock mass the interconnected fractures compose the kinematic porosity which in a two-dimensional section of unit area can be defined as:

$$\Theta_K = e_1 + e_2 + \ldots\ldots + e_n \tag{40}$$

With equal fracture apertures and equal spacing equation (40) becomes:

$$\Theta_K = en \tag{41}$$

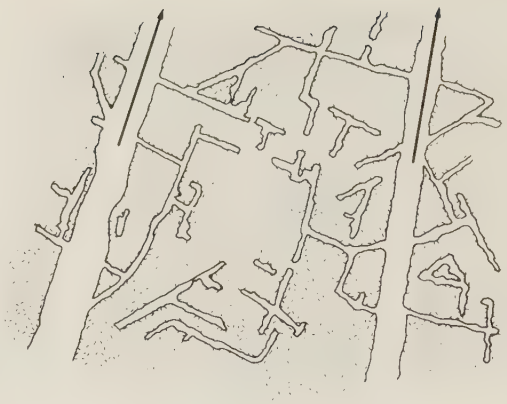


Fig. 40. Schematic representation of the geometric relationship between the different porosities in a rock mass. The arrows denote an arbitrarily chosen direction of water flow within fractures forming the kinematic porosity. The smaller fractures and pores represent the diffusion and residual porosity (Norton and Knapp 1977).

and with three sets of water-bearing joints with equal properties the kinematic porosity will be:

$$\Theta_K = 3en \tag{42}$$

Calculations of the porosity based on measurements in drill-holes will result in different porosity values depending on the penetration of the drill-hole in the different sets. Moreover, the existing joint sets in fact have different fracture properties which is discussed by Parson (1972), Maini (1972), and A. Carlsson (1979).

Snow (1968a) reports a study of fracture spacing, fracture aperture and kinematic porosity, and presents a method of determining the kinematic porosity from water-pressure tests. With equal penetration in three equal joint sets the following relation is obtained:

$$\Theta_K = 2.4\ en \tag{43}$$

A. Carlsson and T. Olsson (1980b) consider different properties in different joint sets. Their work is based on a large number of drillings in a rock mass intersected by four joint sets, one horizontal and three vertical. The fractures included in the different sets have widely divergent properties and the drillings used are both vertical and angular. Thus the vertical drillings mainly penetrate the horizontal joint set while the angular ones pierce the horizontal and one or both of the vertical sets, depending on the direction of the drillhole compared with the strike of the vertical sets. The obtained differences in conductivity between angular and vertical drill-holes are assumed to represent the differences in fracture properties between the joint sets. Their study presents the following relation:

$$\Theta_K = \alpha ne \tag{44}$$

where α = a function depending on the penetration of the drill-holes and of different fracture properties.

The investigation was carried out in the Forsmark area using the values of the α-function

$\alpha = 1.26$ vertical drill-holes
$\alpha = 1.58$ angular drill-holes

These values indicate that in terms of hydraulics, the horizontal joint set is the most important in the area. The addition in kinematic porosity from the vertical sets is very

Table 14. Frequency, apertures and conductivity of the fractures and the kinematic porosity obtained in different parts of the Juktan area.

Location	Test method	Fracture frequency No/m	Fracture aperture μm	Fracture conductivity m/s	Kinematic porosity
Zones of fractured or crushed rock					
Zone A	L	10	74	$2.7 \cdot 10^{-3}$	$2.2 \cdot 10^{-3}$
Zone B	L	10	253	$2.6 \cdot 10^{-2}$	$7.6 \cdot 10^{-3}$
Zone C	L	7	316	$5.0 \cdot 10^{-2}$	$6.6 \cdot 10^{-3}$
Zone D	L	8	248	$3.1 \cdot 10^{-2}$	$6.0 \cdot 10^{-3}$
Zone E	L	6	150	$1.1 \cdot 10^{-2}$	$2.7 \cdot 10^{-3}$
Zone F	L	6	283	$3.9 \cdot 10^{-2}$	$5.1 \cdot 10^{-3}$
Zone G	L	5	216	$2.3 \cdot 10^{-2}$	$3.2 \cdot 10^{-3}$
Blaiksjön	D	6	132–286	$0.9–4.1 \cdot 10^{-2}$	$1.6–3.4 \cdot 10^{-3}$
Homogeneous rock mass					
Blaiksjön	D	2	19–74	$0.2–2.7 \cdot 10^{-3}$	$1.0–3.7 \cdot 10^{-4}$
Renberg	W	1	29–79	$0.4–3.0 \cdot 10^{-3}$	$1.5–4.0 \cdot 10^{-4}$
Tunnel system	L	1	26	$3.3 \cdot 10^{-4}$	$7.8 \cdot 10^{-5}$

L = water leakage, D = water pressure tests in drill-holes, W = well statistics.

small. However, this method calls for both vertical and angular drill-holes.

In the Juktan area there are 24 drill-holes, all vertical, in which water-pressure tests have been made. Considering that the joint system in the Revsund granite consists of one horizontal and three vertical joint sets, the vertical drillings mainly penetrate the horizontal set. Thus it is difficult to evaluate the influence of the vertical sets on the kinematic porosity. The rock masses at Juktan and Forsmark are fairly similar, with a dominant horizontal joint set in the superficial rock mass, which would support the use of an α-factor of 1.26 for the determinations in the Juktan area. The horizontal set in the Forsmark area, however, is more developed (cf. A. Carlsson 1979), which indicates a somewhat higher α-factor at Juktan. For this reason an α-factor of 2 was chosen as being acceptable for determinations based on water-pressure tests. For the calculations based on water leakage, an α-factor of 3 seems reasonable as the tunnel penetration takes place chiefly in the vertical joint sets, and these are the most important for the water-bearing ability of the rock mass at greater depths.

Results

Determinations of the kinematic porosity in Swedish crystalline rocks are rare and few values are reported in the literature. A number of pumping tests are on record but refer to the elastic storage coefficient of the aquifer which does not correspond to the kinematic porosity. A. Carlsson and T. Olsson (1980b), however, report kinematic porosity values between 10^{-6} and $3 \cdot 10^{-3}$ (the corresponding conductivity ranges from about 10^{-11} to 10^{-3} m/s), based on calculations in the Forsmark area. By means of tracer tests L. Carlsson *et al.* (1979) obtained values approaching 10^{-3} in rock masses with a conductivity of about 10^{-6}–10^{-5} m/s. L. Carlsson and T. Olsson (1979a:29) processed results from different areas and report kinematic porosity values of 10^{-5}–$6 \cdot 10^{-4}$ m/s (the corresponding conductivity ranges from $5 \cdot 10^{-11}$–$2 \cdot 10^{-6}$).

The calculations of the fracture apertures, fracture conductivity and kinematic porosity of the rock mass in the Juktan area are summarized in Table 14. Thus the values for kinematic porosity are of the same order of magnitude as those presented above. The values for fracture frequency reported in Table 14 were variously obtained depending on the method of determining the hydraulic conductivity. Calculations based on water-pressure tests use the fracture frequency obtained from the core-mapping. In the calculations based on water leakage and well statistics the fracture frequency was assessed within the zones in the tunnel system. These estimates refer to both the water-bearing zones and the intervening intact rock mass.

The fracture frequency varies little in different parts of the tunnel. The highest reported frequencies were recorded in zones A and B near Lake Renbergssjön, where the rock is of poor quality as manifested in very extensive tunnel supports there. The other water-bearing zones present a similar frequency with about 6 fractures/m. The frequency in the homogeneous rock mass averages about 1 fracture/m. The differences within a single zone or throughout the rock mass are considerable, but could not be determined.

The calculated fracture apertures vary greatly between different parts of the tunnel; the widest apertures in zones of fractured or crushed rock far exceed 100 μm while in the homogeneous rock mass they usually fall short of 50 μm. This difference between the rock mass and the zones of weak rock also appears in the values for fracture conductivity and kinematic porosity.

The variations in the different parameters are probably even greater than reported. If determination methods allowing a selective testing of individual fractures are used, the variation will be even greater. This was stated by L. Carlsson and T. Olsson (1979a) and A. Carlsson and T. Olsson (1980b).

Comparison of the values in Table 14 with those reported in the literature (Snow 1968a:82, Norton and Knapp 1975:920, L. Carlsson and T. Olsson 1979a:29, L. Carlsson *et al.* 1979 and A. Carlsson and T. Olsson 1980b) reveals that the values from the rock mass in the

Juktan area are about the same as these from other areas of crystalline rocks. But the values from zones of fractured or crushed rock are generally somewhat higher in Juktan compared with studies in other areas.

Groundwater balance in the Juktan area

The groundwater balance was calculated in order to assess the reliability of the values obtained for the hydraulic properties of the aquifers, and for the recharge due to precipitation percolation. The hydraulic conductivity of the aquifers is valid only where the observations were made, the representativity being dependent on the determination method and the hydraulic properties. Thus the validity and accuracy at the regional level must be estimated. The groundwater recharge is here based mainly on recordings from Stensele, outside the actual investigation area, so that the representativity of the recharge for the Juktan area must be determined.

The drainage due to tunnelling offers a further opportunity to determine the reliability of the obtained parameters. The groundwater balance is here determined both for undisturbed and disturbed hydraulic conditions.

Different methods and different approaches were adopted for the calculation of the groundwater balance. First and foremost analytical calculations were executed, but also numerical methods were applied to determine the balance under conditions disturbed by the construction works. The use of several approaches in the calculations allowed for a multiple check of the obtained values of the parameters.

Defining the groundwater balance

According to Brown *et al.* (1972:5.1.2) the balance for the groundwater régime for the long-term period may be defined as:

$$P_P + P_S + Q_R + P_A = E + Q_S + Q_D + Q_A \qquad (45)$$

where P_P = recharge due to precipitation percolation
P_S = recharge from surface water sources
Q_R = groundwater inflow
P_A = artificial recharge
E = evapotranspiration from the aquifer
Q_S = discharge to surface water sources
Q_D = groundwater discharge
Q_A = artificial discharge

The groundwater balance requires that the equation be on balance, any difference being explained by changes in groundwater storage.

Regarding a natural, undisturbed groundwater-system where no groundwater inflow takes place, the equation states that the recharge from the surface discharges via groundwater outflow (except for the evapotranspiration). The groundwater outflow depends on the hydraulic properties of the aquifer which is always valid, whether the groundwater discharges to a surface water system, or as groundwater discharge.

In the Juktan area there are two aquifers in which the recharge and discharge are composed of different processes. Assuming little evapotranspiration from the aquifer (most evapotranspiration utilizes water stored in the aerated zone), the following relation is valid for the aquifer system:

$$P_T = Q \qquad (46)$$

where P_T = recharge to the soil aquifer
Q = groundwater outflow

The groundwater outflow may however be subdivided depending on various groundwater flows in different aquifers:

$$Q = Q_T + Q_R \qquad (47)$$

where Q_T = groundwater outflow in the till aquifer
Q_R = groundwater outflow in the rock aquifer

For the till aquifer the following relation is valid:

$$P_T = Q_T + P_R \qquad (48)$$

where P_R = recharge to the rock aquifer

In these equations the terms Q_T and Q_R are determined by:

Horizontal hydraulic conductivity in the till and the rock aquifers

Horizontal hydraulic gradients in the respective aquifers

The recharge to the rock mass is determined by:

Vertical conductivity of the till and the rock aquifers

Difference in head between the aquifers

However a considerable vertical groundwater flow may exist in the rock aquifer, which may be defined as recharge to underlying layers of the rock mass. These definitions are valid for recharge areas, while the processes in discharge areas cause negative sign on the included parameters.

Aquifer properties and groundwater recharge. Table 15 summarizes the values obtained for the hydraulic conductivity and groundwater recharge for the main aquifers.

The conductivity of the till soil was determined by means of laboratory tests as about $2 \cdot 10^{-7}$ m/s. This value supposedly represents the vertical conductivity, while the horizontal conductivity is thought to be higher by reason of dominating horizontal structures. Experience from other investigations shows that conductivity values obtained by laboratory tests on disturbed samples tend to be about one decade lower than the actual horizontal conductivity of the natural till. The differences between laboratory and *in situ* tests, however, are due to existing structural discontinuities and their hydraulic connections, which implies that the conductivity on a regional scale will be governed to a great extent by the interconnections between the discontinuities. Therefore the actual horizontal hydraulic conductivity of the till aquifer, will fall in the range $2 \cdot 10^{-7} - 2 \cdot 10^{-6}$ m/s, but on the regional level probably not as high as the upper limit of the range. A reasonable value would be about $5 \cdot 10^{-7}$ m/s.

The hydraulic conductivity in the superficial rock mass was determined as about 10^{-6} m/s for the rock mass as a whole. In reality the horizontal conductivity of the homogeneous rock mass is much lower, down to 10^{-9} m/s are recorded. Indeed zones of fractured and crushed rock are more conductive, up to about 10^{-4} m/s.

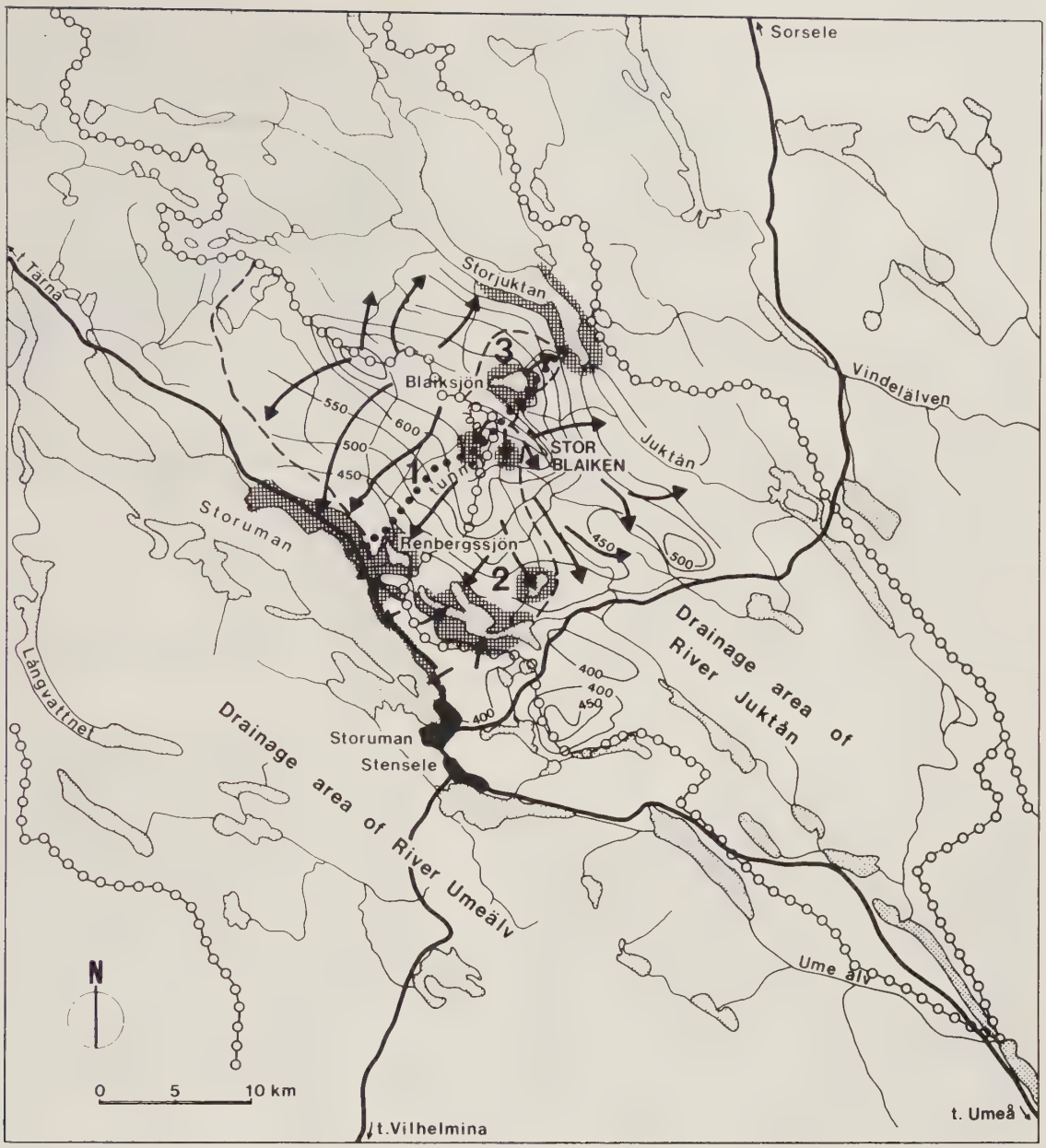


Fig. 41. Groundwater levels, flow directions and discharge areas in regional scale in the Juktan area. Mount Storblaiken acts as a water divide between the drainage areas of the Juktån and Umeälv rivers. 1, 2 and 3 represent local drainage areas.

Table 15. Hydraulic conductivity and recharge values of the main aquifers at Juktan.

Aquifer	Hydraulic conductivity Vertical m/s	Horizontal m/s	Recharge mm/year
Till soil	$2 \cdot 10^{-7}$	$5 \cdot 10^{-7}$	115
Bedrock	$1-5 \cdot 10^{-6}$	$1 \cdot 10^{-6}$	
Rock mass	$1-10 \cdot 10^{-7}$	$1-10 \cdot 10^{-7}$	7
Tectonic zone	$4 \cdot 10^{-5}$	$4 \cdot 10^{-5}$	20

Soundings of the groundwater levels in the Juktan area indicate that the levels in the rock mass and in the till cover are in fairly close agreement. During periods with heavy recharge, the groundwater in the soil seems to react faster than in the rock mass and a difference in head of a few cm may then occur. As a rough approximation based on individual measurements, the groundwater in the till aquifer is estimated to be about 0.5—5 mm above that in the rock aquifer. This was observed in the recharge areas in the vicinity of Lake Blaiksjön. In discharge areas an opposite gradient is developed with the highest water pressure in the rock mass. However, no measurements are available to verify this statement. The lateral hydraulic gradient along the tunnel-system is about $2 \cdot 10^{-2}$ m/m between Mt Storblaiken and Lake Renbergssjön, as seen in Fig. 41.

Regional distribution of groundwater flow. A sketch map of the groundwater levels visualizes the flow pattern within the area (Fig. 41). Due to sparse information from groundwater records, the existing lakes served as reflections of the groundwater level. The advantage of using lakes is that the regional pattern will be more developed and the local system overridden. A similar principle was followed earlier by Lindblom *et al.* (1977:37) in two different areas of southern Sweden. This method is only valid if the conductivity of the aquifer and the bottom of the lake are about the same. Lakes situated on conductive formations do not necessarily represent the groundwater level, as emphasized by Hörnsten *et al.* (1977) concerning the lakes at Mt Billingen in southern Sweden.

On average, about 80 per cent of the land area is made up of recharge and about 20 per cent of discharge areas. In general, the great river valleys, the Umeälv and Juktån, and the depressions in which Lakes Renbergssjön and Gunnarnsjön are situated, comprise the main discharge areas. This is valid for the regional groundwater flow, the local flow systems dividing the regional recharge areas in local recharge and discharge areas depending on local variations in the small-scale morphology.

Groundwater flow within and between the aquifers. The proportion of groundwater which discharges within each aquifer may be determined by means of the values obtained for the hydraulic conducitivity and the lateral hydraulic gradients. Furthermore it is possible to determine the groundwater recharge at different depth in the rock mass. With a value of $5 \cdot 10^{-7}$ m/s for the horizontal hydraulic conductivity of the till aquifer, and a hydraulic gradient of $2 \cdot 10^{-2}$ m/s, the groundwater discharge in the till may reach as much as 315 mm/year. This value is much higher than the calculated recharge to the till, suggesting that either the conductivity is too high or the recharge too low. Considering the high infiltration capacity of the till (200—300 mm/h) it is obvious that the total amount of precipitation available may infiltrate. But the discharge of this quantity of groundwater is probably confined to a limited recharge and discharge area. Nevertheless it is possible although less probable that the conductivity of the natural till soil is overestimated. A conductivity of $1.6 \cdot 10^{-7}$ m/s, which is in accordance with the laboratory results, will give a discharge in the till aquifer in balance with the calculated recharge minus the recharge to the rock aquifer (about 100 mm/year).

Using the equation for leaky aquifers, given by Huisman (1972:20), the groundwater flow from the till cover to the rock mass was determined:

$$P_R = k' \frac{\Delta h}{H'} \qquad (49)$$

where P_R = recharge to the rock mass (m/s)
k' = vertical conductivity of the till cover or the rock mass (m/s)
Δh = difference in head between the aquifers (m)
H' = water-bearing thickness of the till formation (m)

The aforesaid values of the conductivity and water pressure differences in recharge areas give a groundwater flow from the till into the rock aquifer of 1—6.3 mm/year. During periods with increased pressure differences the flow also increases and may momentarily be very heavy.

The lateral groundwater flow through the rock mass amounts to about 12 mm/year, based on a conductivity of $2 \cdot 10^{-8}$ m/s and a gradient of $2 \cdot 10^{-2}$ m/m. Hence the demand for continuity is fulfilled. The values obtained correspond closely to those presented earlier for the recharge to the rock mass. Calculations based on water level response gave a recharge of 7 mm/year as being the recharge to the rock, which is of the same order of magnitude as the values obtained in this section. The hydraulic conductivity of the rock mass decreases with increasing depth, and the discharge will follow suit.

To determine the discharge at different depths, calculations of the relative discharge were made on the basis of statistical relations presented above.

The gradient of the depth dependence of the conductivity is set at −2.1. According to L. Carlsson and T. Olsson (1979a:53) the pressure gradient based on numerical calculations also decreases with increasing depth. Analysis of their data for a recharge area yielded a relation for the gradient versus depth of

$$I = 10^{-(1.4 \cdot 10^{-3} D + 0.02)} \qquad (50)$$

where I = hydraulic gradient
D = depth below groundwater level (m)

It must, however, be noted that the gradient versus depth varies greatly, according to the part of the aquifer represented, and also that the gradient depends on the hydraulic conductivity of the aquifer. This equation only illustrates the relation to depth and was used merely to visualize the variation in flow versus depth.

Combining these depth dependences makes it possible to determine the discharge at different depths and offers an

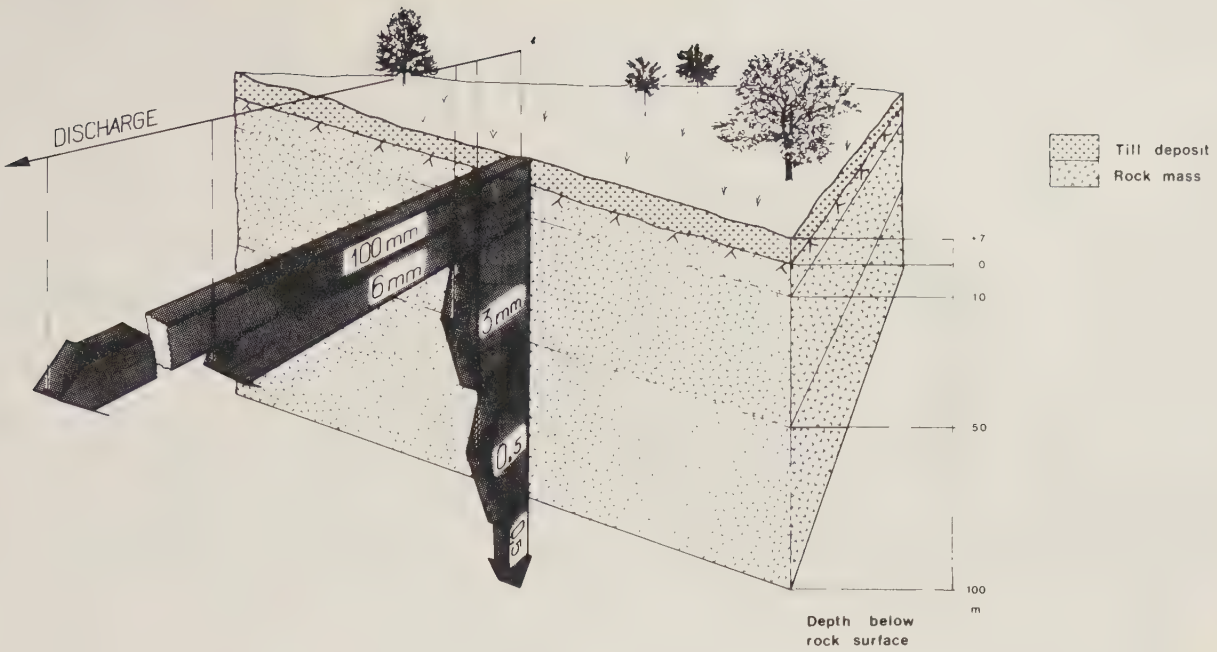


Fig. 42. Groundwater flow in the main aquifers and flow versus depth in the rock aquifer. Note that the arrow representing the groundwater discharge from the till cover is broken.

opportunity to calculate the transformation velocity of the groundwater. The results are shown in Table 16. Thus the majority of the groundwater flow is confined to the superficial rock mass, i.e. about 60 per cent of the total groundwater flow occurs in the uppermost 10 m. Below the 200 m-level less than 2 per cent of the groundwater flow takes place.

Figure 42 illustrates the distribution in groundwater flow between the aquifers and within the rock mass at different depths.

Influence of the topography on the groundwater situation

Analytical calculations. The groundwater flow in an aquifer is governed by the hydraulic conductivity of the aquifer and the predominant pressure conditions. Differences in hydraulic head create the forces necessary for the flow. The difference in head is variable and the pressure gradient is generally greater in the superficial rock mass than lower down as shown by L. Carlsson and T. Olsson (1979a:49). At the groundwater level the hydraulic gradient is equal to its inclination.

Table 16. The distribution of groundwater flow at different depths in the rock mass.

Depth (m)	Groundwater flow in per cent of total flow
0–10	59
10–20	18
20–30	8
30–50	7
50–100	4
100–150	2
150–200	1
>200	1

As a result of the humid climate in Sweden coupled with the relatively low conductivity of the rock, a greater volume of water is continuously available for recharge than discharges as groundwater. The groundwater level therefore is mainly governed by the topographical conditions. The groundwater level is normally situated very near to the surface and closely follows its undulations as shown by Gustavsson (1968:5) and Hörnsten *et al.* (1977:65). Formations with high hydraulic conductivity tend to flatten the topographically induced groundwater level. Such formations include eskers and major fracture zones in rock, where the groundwater level is independent of the morphology. The levelling effect of fracture zones was pointed out by C. Axelsson and L. Carlsson (1979b) and L. Carlsson and T. Olsson (1979a:39).

The effect of the topographically induced groundwater level decreases with increasing depth. Different hydraulic heads therefore exist at different depths, and according to whether this vertical distribution in head increases or decreases, discharge or recharge areas occur. Larger zones of fractured or crushed rock intersecting the rock mass will act as draining structures, which implies that the surrounding rock mass will act primarily as recharge areas. But tectonic zones too may leak. This was discovered by L. Carlsson and T. Olsson (1979a:53) who discussed the importance of fracture zones from calculations based on the finite element method.

Analytical calculations were carried out along a longitudinal profile through the area between Lakes Storuman and Storjuktan, in order to illustrate the effects of the topography and determine the reliability of the obtained hydraulic parameters. The profile is located on the line of the tunnel and the groundwater flow is thought to

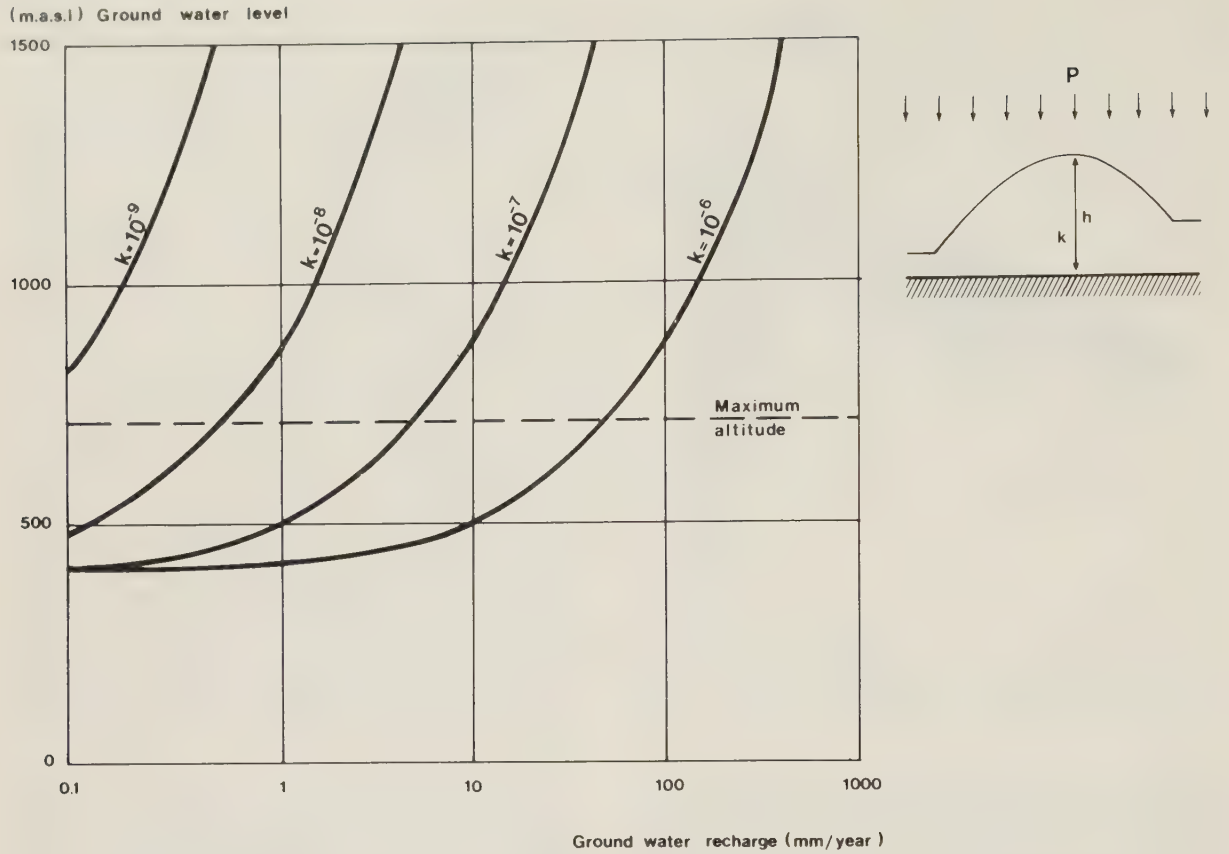


Fig. 43. Maximum groundwater level in the Juktan area as a function of the hydraulic conductivity and the infiltration recharge.

take place in the plane of the profile, i.e. the profile follows the groundwater circuits. The difference between this approximation and the real groundwater flow seems to be small and presumably negligible (cf. Fig. 41).

The requirements are stated in the following assumptions:

The groundwater level is congruent to the ground surface, which also limits its maximum rise. Surface run off occurs when the groundwater level seeks to surmount the ground surface.

The majority of the groundwater circulation takes place in the superficial rock mass. No groundwater flow is assumed to occur below an purative boundary at an altitude of +300 m above sea level, i.e. 50 m below Lake Storuman which is the lowest point in the profile.

The hydraulic conductivity of the rock mass is presumed constant in the water-bearing part of the rock mass, i.e. between the rock surface and the lower boundary at +300 m above sea level. This presumption is in fact inaccurate, but since the major part of the groundwater flow is confined to the uppermost part of the rock mass the inaccuracies arising from this assumption are of minor importance.

The levels of the lakes in the profile are assumed constant, and governing the groundwater levels in the rock mass.

The groundwater recharge to the rock is presumed constant throughout the profile.

No groundwater flow takes place perpendicular to the profile, except those points at wich boundary conditions are stated.

Assuming that the groundwater levels in the profile are governed by the levels of Lakes Storjuktan and Storuman, by the hydraulic conductivity of the rock mass and finally by the recharge to the rock mass, the profile can serve as the starting point for a study of the groundwater levels in the profile. The altitudes of the lakes are +410 m and +350 m above sea level respectively, i.e. 110 m and 50 m above the stated lower boundary at +300 m. The lakes are about 20.4 km apart. Combination of Darcy's law and the law of continuity gives an equation which describes the altitude of the groundwater level as a function of the conductivity and recharge (cf. Huisman 1972:30).

$$h_{max}^2 = \frac{P}{4k} L^2 + \left(\frac{h_2^2 - h_1^2}{L}\right) \frac{k}{4P} + \frac{h_2^2 + h_1^2}{2} \qquad (51)$$

where h_{max} = maximum groundwater level (m)
k = hydraulic conductivity of the rock mass (m/s)
P = groundwater recharge (m/s)
L = distance between the hydraulic boundaries (m)
h_1, h_2 = height of the hydraulic boundaries above the lower boundary (m)

This relation is graphically shown in Fig. 43. Thus a higher recharge is required when the hydraulic conductivity is higher in order to maintain a certain groundwater level. Consequently, with a lower conductivity value a corresponding lower recharge is required.

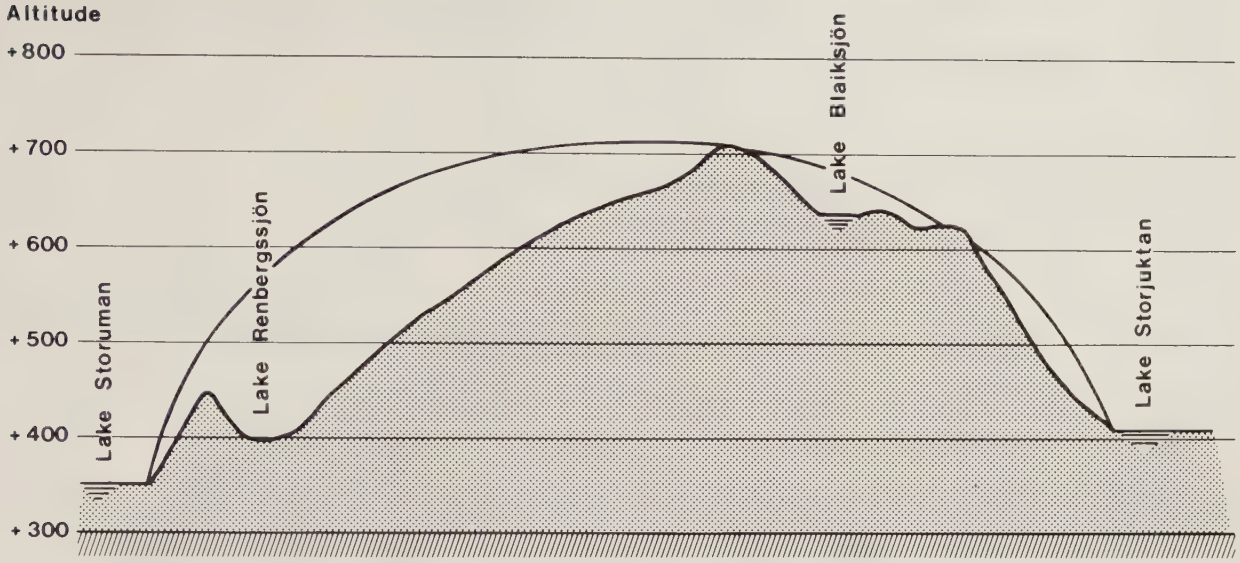


Fig. 44. Calculated groundwater levels in the profile. Boundary conditions at Lakes Storuman and Storjuktan and at Mount Storblaiken.

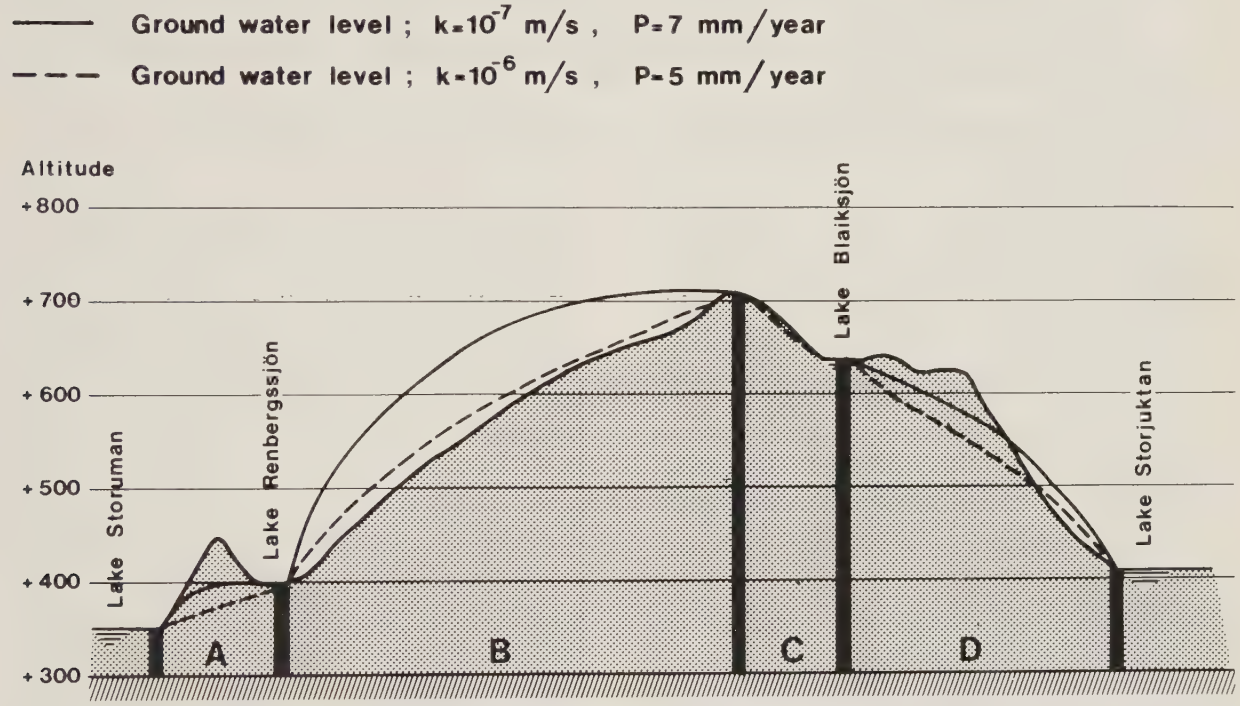


Fig. 45. Calculated groundwater levels in the profile. Boundary conditions at Lakes Storuman, Storjuktan, Blaiksjön and Renbergssjön and at Mount Storblaiken.

However, this simple approach regarding the ground water balance of the profile is only approximate, as boundary conditions in the form of fixed levels also exist within the profile. These boundaries are e.g. Lakes Blaiksjön and Renbergssjön, and also the highest topographical level in the profile, Mt Storblaiken. A groundwater situation as shown in Fig. 44 is obtained considering the level of Mt Storblaiken and ignoring the lakes. This situation prevails with any conductivity and recharge values from the diagram in Fig. 43 corresponding to the maximum altitude.

Figure 44 shows that only the level of Mt Storblaiken is satisfied with this approach, while the groundwater level in the rest of the profile is situated far above the ground surface and even above Lakes Blaiksjön and Renbergssjön.

Using the levels of Lakes Blaiksjön and Renbergssjön as internal boundary conditions a new presumption must be included, namely that no groundwater flow pass the boundaries from one part of the profile to another. Each part limited by the boundaries must be calculated separately. These new internal boundary conditions permit groundwater flow perpendicular to the profile at the boun-

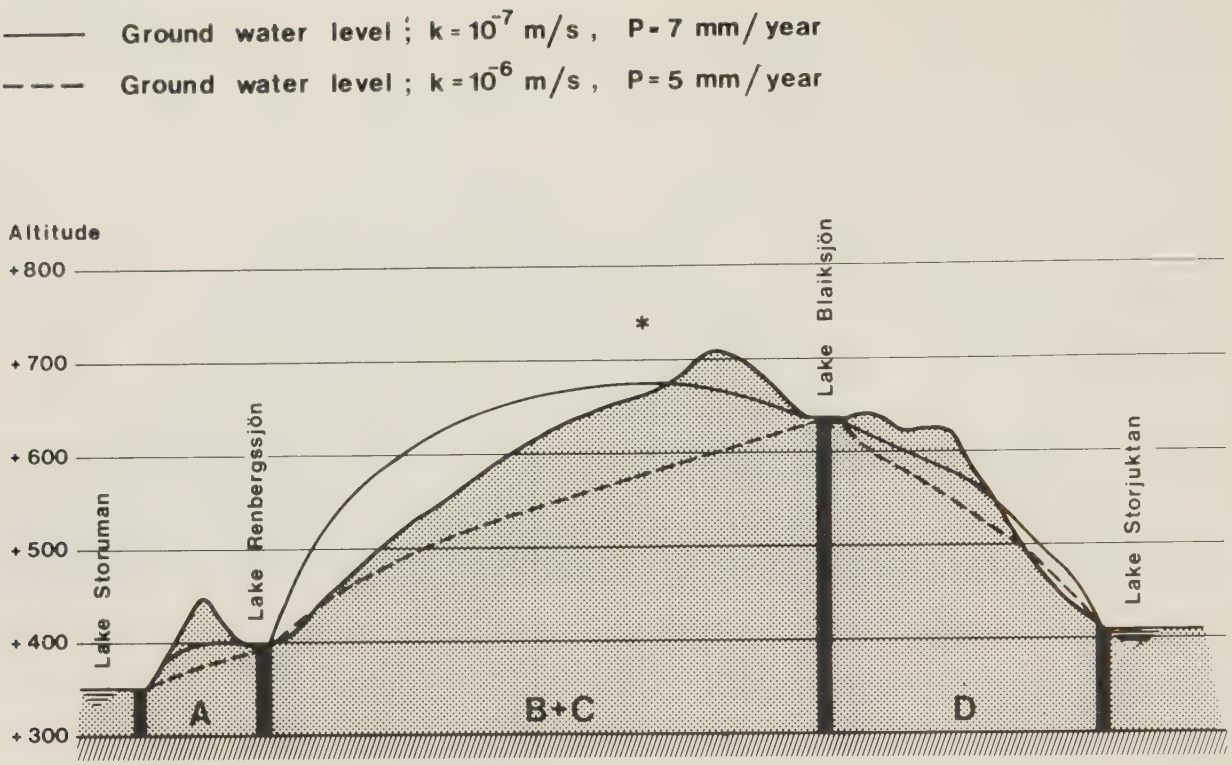


Fig. 46. Calculated groundwater levels in the profile. Boundary conditions at Lakes Storuman, Storjuktan, Blaiksjön and Renbergsjön. The position of the ventilation shaft is indicated by *.

daries, i.e. ground water recharge or discharge may occur to maintain the stated levels.

The situation resulting from these internal boundary conditions is shown in Fig. 45. The profile is divided into four sections, designated A, B, C and D the groundwater level in each being calculated for two pairs of conductivity/recharge values ($k = 10^{-7}$ m/s, P = 7 mm/year and $k = 10^{-6}$ m/s, P = 5 mm/year). A comparison of this groundwater situation with that in Fig. 44 shows that the calculation in Fig. 45 is in much better agreement with the topographical conditions. However the groundwater level obtained in part B of the profile is far above the ground surface, which applies to both pairs of conductivity/recharge.

To maintain the groundwater level at the highest topographical boundary, a high natural recharge must occur which is solely dependent on infiltration of precipitation. To control this condition, a calculation was made in which the internal boundary at Mt Storblaiken was removed. Parts B and C were thereby considered as one unit. The groundwater situation obtained is shown in Fig. 46. The result of this treatment shows that only the pair with the lower conductivity and higher recharge can maintain the groundwater level at the highest topographical levels. But in this calculation too the groundwater level in area B is far above the ground surface. The calculation with the second pair (high conductivity and low recharge) results in a less reliable groundwater situation with the groundwater level situated more than 100 m below the ground surface.

Discussion of the results. The results obtained in the calculations give a groundwater situation which in some cases corresponds fairly well with the stated presumptions. Of the conductivity/recharge values used, one pair of parameters ($k = 10^{-7}$ m/s and P = 7 mm/year) was chosen from the aforesaid determinations. The other pair of values ($k = 10^{-6}$ m/s and P = 5 mm/year) was selected because under certain circumstances they resulted in a better correspondence between the calculated model and the actual conditions. The latter pair gives an apparently better correspondence in some of the calculations, i.e. the calculations with a boundary condition set at Mt Storblaiken. However, this pair of values cannot maintain the groundwater level at this point as illustrated by Fig. 46, which indicates either too high a value of the conductivity or too low a recharge.

But the results based on the former pair of parameters show closer correspondence between the calculations and the actual groundwater condition, although the groundwater level in some parts of the profile is much too high. This is particularly true of part B. Yet to maintain the actual levels in the profile it is essential to use a conductivity/recharge relation of at least this magnitude ($k = 10^{-7}$ m/s, P = 7 mm/year). In parts A and D not even this pair can create groundwater levels in accordance with the reality.

The reason for the discrepancy between the calculated model and the topographical conditions probably consists in diverse circumstances. The presumptions for the calculations stipulate that the rock mass between the internal boundaries be homogeneous with constant conductivity, but also that the groundwater recharge be constant throughout the area.

As stated above, the rock mass is not homogeneous, indeed it is inhomogeneous with a hydraulic conductivity varying over a wide range. Tectonic zones of fractured and

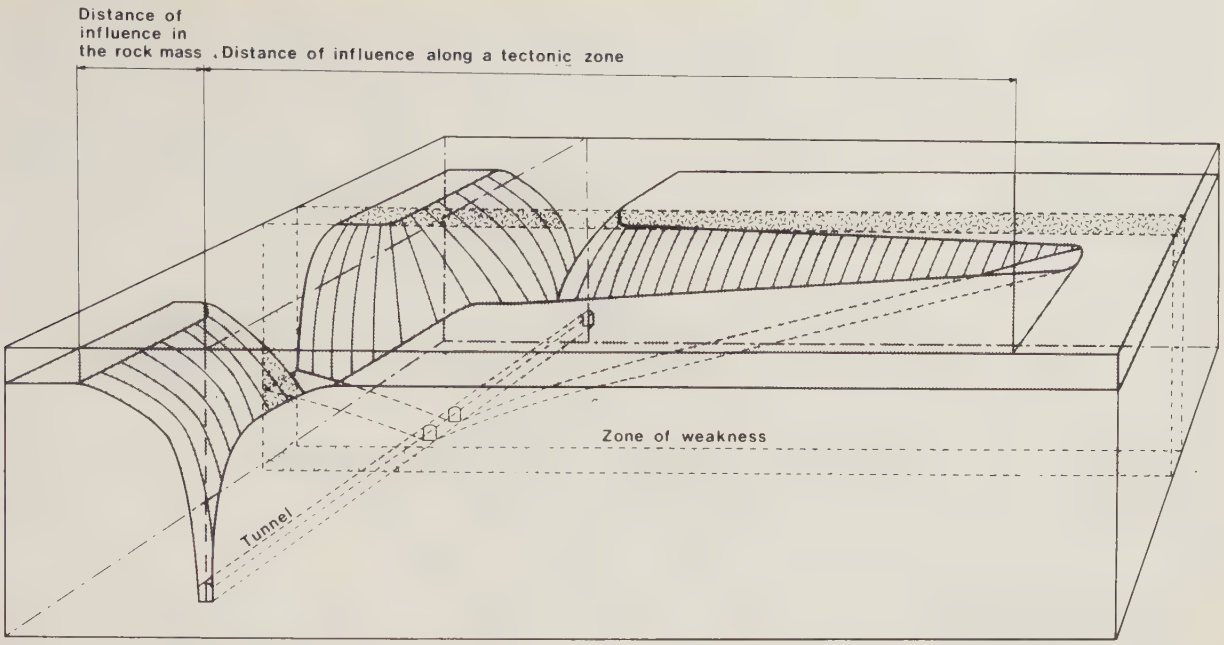


Fig. 47. Schematic illustration of the groundwater drainage along a tunnel. The distance of influence is different in the homogeneous rock mass and in the fracture zones (modified after A. Carlsson and T. Olsson 1978b).

crushed rock intersect an otherwise fairly homogeneous rock mass. In consequence a substantial groundwater flow diverging from the profile may occur, which gives rise to additional internal boundaries of complex nature. The presence of tectonic zones is known from parts B and D (cf. Fig. 6 and 21 and the description of the groundwater leakage).

Varying pressure conditions within the profile indicate that the recharge too will vary. In the calculations the recharge is presumed constant throughout the profile. However the actual recharge is probably greater in the upper parts of the recharge areas and will gradually decrease downwards and finally become a negative recharge (discharge) in the discharge area.

The high groundwater levels obtained in some parts of the profile are in agreement with geological indications. The map of the Quaternary deposits (Fig. 8) shows large areas along the tunnel system composed of till soils covered by thin layers of peat, which signifies moist conditions. These areas are chiefly located in the vicinity of Lake Blaiksjön, and between Mt Storblaiken and Lake Renbergssjön. In these areas the analytical calculations also resulted in high groundwater levels.

Artesian groundwater conditions were found during the drilling of a ventilation shaft for the tunnel. This drill-hole is located south of Mt Storblaiken at the point marked in Fig. 46. It is situated near the highest point of the profile and even at this altitude the groundwater level rose above the ground surface. During drilling the drill-hole overflowed with a capacity of about 9 l/s (Fredriksson 1978:2).

The thin peat cover and the artesian groundwater conditions confirm the results of the calculations and thereby the magnitude of the hydraulic conductivity and the recharge. However, the values of these parameters are complex in nature. The hydraulic conductivity (10^{-7} m/s) is representative of a rock mass composed both of homogeneous rock poor in fractures and of densely fractured zones, which implies that differentiation will result in lower conductivity for the homogeneous rock mass and higher for zones of fractured and crushed rock. The groundwater recharge can be regarded as a mean value for the entire area, which suggests that the recharge in recharge areas is higher. Considering the aforesaid distribution between recharge and discharge areas, the actual recharge in recharge areas would on average be between one or two times greater than the assumed 7 mm/year.

The influence of the tunnelling on the groundwater balance

Distance of influence

The area influenced by the groundwater drainage due to the tunnelling is crucial for the groundwater balance under disturbed conditions. The recharge which takes place within this area will leak into the construction and discharge by pumping. The size of the area of influence is dependent on various factors, such as the conductivity of the aquifer, construction depth, existing zones of fractured and crushed rock and so forth. In high-conductive aquifers great quantities of groundwater may leak into the construction, multiplying both the leakage and the area of influence. In low-conductive formations these effects will be small and local.

A number of papers deal with the area of influence in connection with drainage due to subterranean constructions. Gustavsson *et al.* (1970) presented a study in which the distance of influence is assumed to depend only on the depth of the tunnel below the groundwater level. This work states that 50 per cent of the total groundwater leakage originates from a zone with a width twice the depth of the tunnel and 80 per cent from a zone 6 times as wide. The

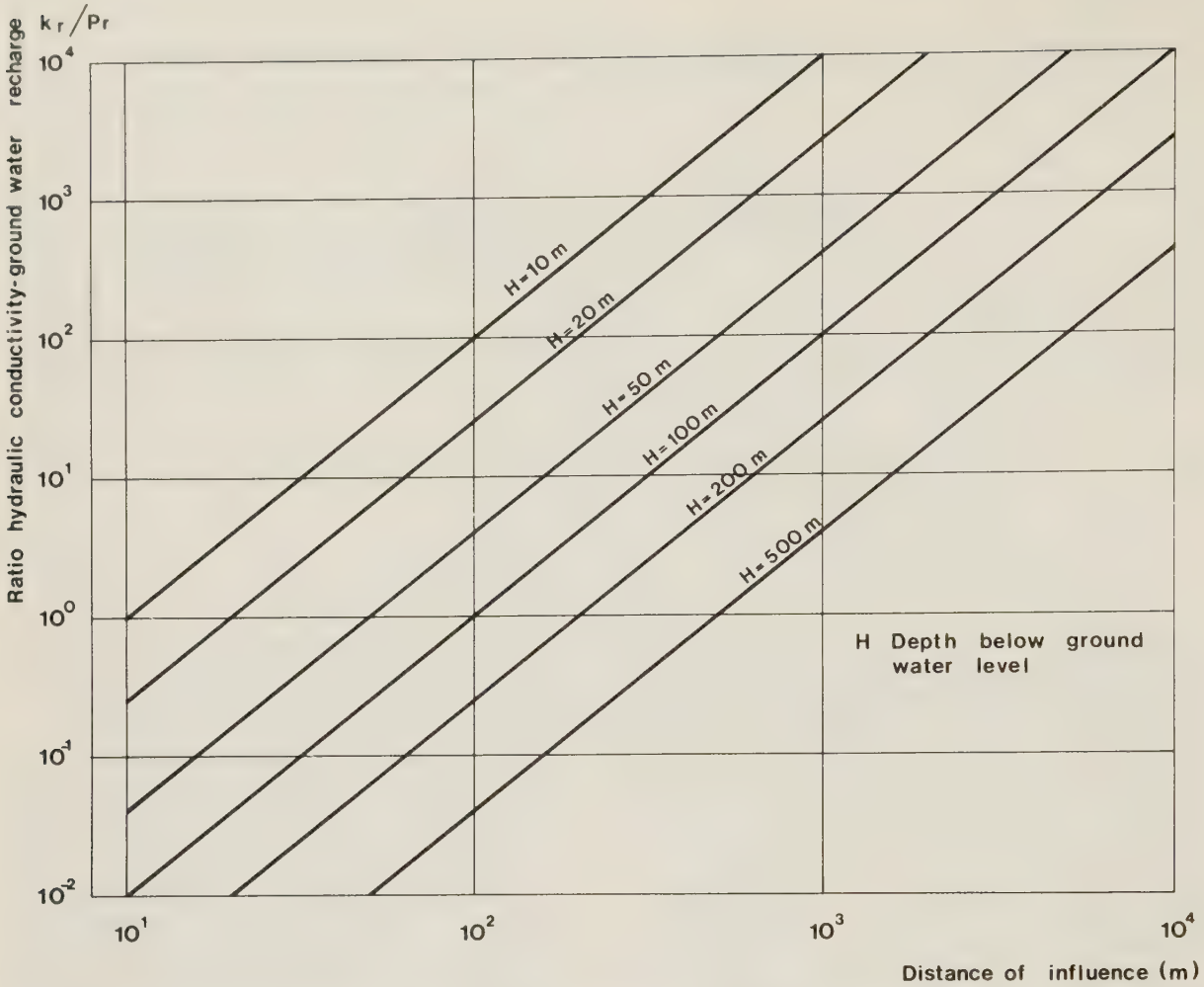


Fig. 48. The distance of influence in the homogeneous rock mass as a function of the hydraulic conductivity and the recharge for different depths below the original groundwater level.

latter zone is thought to be equal to the distance of influence. However, this method is a very rough approximation as pointed out by e.g. A. Eriksson (1975:47) and A. Carlsson and T. Olsson (1978b:14).

A model which describes the distance of influence as a function of the conductivity, the recharge in terms of leakage and the construction depth was presented by A. Carlsson and T. Olsson (1978b). This method defines the distance of influence as the distance at which the groundwater decline exceeds 1 m. This definition is chosen as being the amount of groundwater decline at which the drawdown exceeds the natural fluctuations of the groundwater level. Drainage due to conductive zones of fractured and crushed rock is also taken into consideration.

The hydraulic conductivity of a rock mass is, however, not constant, for parts or zones with completely different conductivity alternate. High conductive zones usually intersect an otherwise fairly homogeneous rock mass. These zones give rise to an influence at much greater distances than expected from the conductivity of the homogeneous rock mass. This condition is illustrated in Fig. 47 with different distances of influence in the homogeneous rock mass and the conductive zone.

The distance of influence along highly conductive zones is dependent not only on the conductivity, the recharge and the depth below the ground water level of the zone, but also on its width and on the conductivity of the surrounding rock mass. Drawdown due to drainage along zones may amount to several km which may cause damages in areas situated far from the tunnel.

L. Carlsson *et al.* (1979b) present a calculation model similar to the former, but with an important difference in the definition of the distance of influence. In this model the distance of influence is the actual distance of influence, i.e. the distance where no influence on the groundwater system is to be found. This method therefore gives the effective precipitation area influenced by the drainage due to the tunnelling. In addition the natural groundwater flow can be considered. This method uses a constant recharge throughout the area influenced by the construction.

The principles of the last two methods were also used to calculate the distance of influence at certain constructions (L. Carlsson *et al.* 1977:31, Hörnsten *et al.* 1977:128, Nordberg *et al.* 1977:69 and Baeckström *et al.* 1978:24).

All reported methods for determination of the distance of influence imply that the drawdown at a subterranean construction is solely dependent on the depth below the

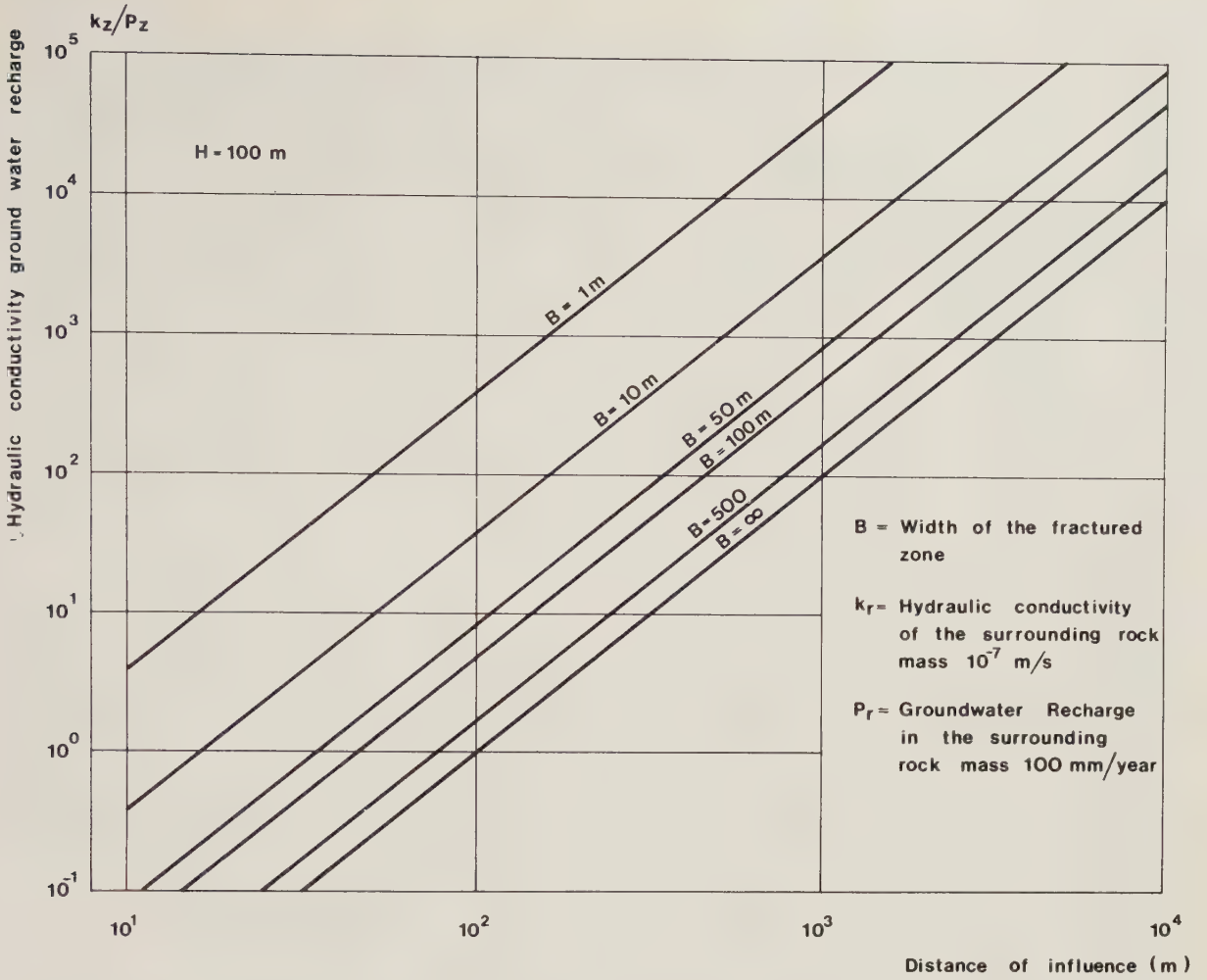

kz/Pz
Hydraulic conductivity ground water recharge
H = 100 m
B = 1m
B = 10m
B = 50m
B = 100m
B = 500
B = ∞
B = Width of the fractured zone
kr = Hydraulic conductivity of the surrounding rock mass 10⁻⁷ m/s
Pr = Groundwater Recharge in the surrounding rock mass 100 mm/year
Distance of influence (m)

Fig. 49. The distance of influence along zones of fractured and crushed rock as a function of the zone-conductivity and recharge for different widths of the zones at a construction situated 100 m below the original groundwater level.

groundwater level. In formations with low hydraulic conductivity, however, the fractures are not capable of draining the rock mass above the construction. In this case a hanging groundwater storage is developed above the rock chambers and tunnels, which may have a groundwater level in agreement with the natural level. But since a large part of the groundwater flow in such storage is directed vertically and takes place under higher gradients than usually exist in undisturbed aquifers, the hanging aquifer will have greater level fluctuations than the natural aquifer.

The existence of this hanging aquifer was demonstrated by measurements above the Stripa mine in central Sweden (Gale and Witherspoon 1978).

Distance of influence in the Juktan area. To determine the distance of influence of the Juktan tunnels, calculations were made according to the same principles as those used in L. Carlsson *et al.* (1978:74), but somewhat modified. Figures 48 and 49 show the distance of influence for the homogeneous rock mass and the conductive zones.

Figure 48 visualizes the distance of influence in the homogeneous rock as a function of its conductivity and recharge for different depths below the groundwater level. Figure 49 presents the distance of influence along zones of fractured and crushed rock as a function of the zone-conductivity and recharge for different widths of the zone at a construction situated 100 m below the original groundwater level. The conductivity of the surrounding rock mass is assumed constant at 10^{-7} m/s. The distance of influence is dependent on the width of the zone since the zone will also drain the surrounding rock mass, which will result in a apparently increased recharge into the zone. The effect of the increased recharge varies with the width of the zone.

The distance of influence also depends on the natural groundwater flow in the area. This is of vital importance if the direction of the tunnel is perpendicular to the flow as shown by L. Carlsson *et al.* (1979b:76), since the distance differs according to whether the influence is upstream or downstream from the tunnel.

In the Juktan area the tunnels and the natural groundwater flow are almost parallel, which only slightly changes the distance of influence. In the Juktan area with a recharge under drained conditions of about 15 mm/year in the homogeneous rock mass and 150 mm/year in the tectonic zones the distance of influence will be as follows:

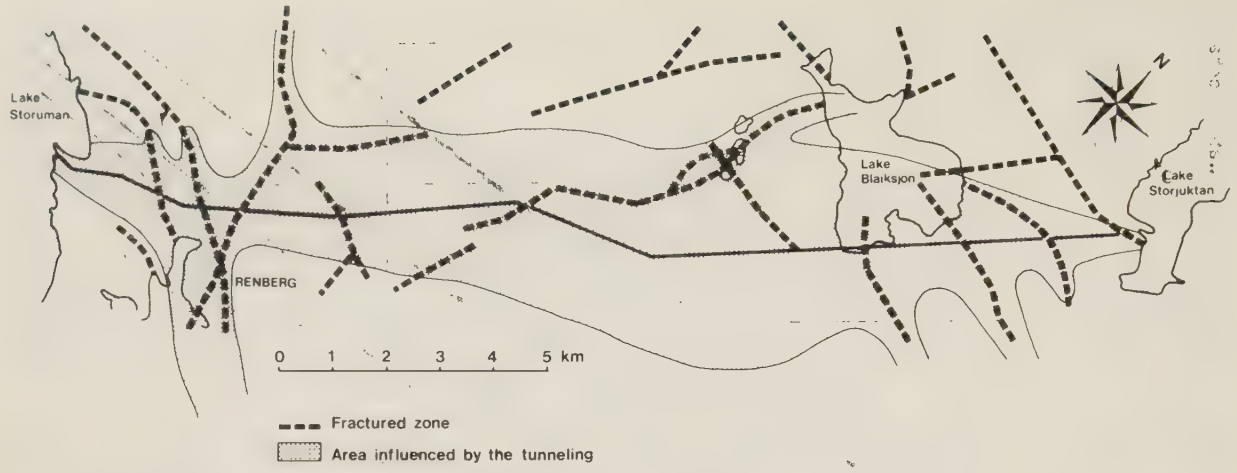


Fig. 50. The area influenced by the groundwater drainage due to the tunnelling.

1,000 m on average in the homogeneous rock mass

1,500–10,000 m along tectonic zones

These are approximate values; indeed the distance of influence will vary with depth below the groundwater level and with width of the tectonic zone.

The calculated area influenced by the tunnelling is shown in Fig. 50. The total area influenced is estimated to about 50 km^2.

A comparison of distances of influence between the tunnel-systems of Juktan and Himmerfjärden in southern Stockholm is presented in A. Carlsson and T. Olsson (1978b:16). This comparison shows that the calculation model presented therein yields reliable results which are verified by data from both areas (A. Carlsson and T. Olsson 1978a:382, 1979d:266, A. Eriksson 1975:4 and Sund *et al.* 1977:56).

The huge distances of influence along the zones of fractured and crushed rock obtained in the Juktan area are probably unique for Sweden. Distances of influence of about the same magnitude are not reported from any subterranean construction in Sweden. But internationally this extent of influence is fairly common, although generally in other rock types. In Hungary Schmieder (1971:433) found a distance of influence of about 10 km in a coal mine situated 40 m below the groundwater level, and this value is valid for the homogeneous rock mass. L. Carlsson *et al.* (1979b:72) report a survey of reported distances of influence both from Swedish and non-Swedish constructions which show that huge distances may occur, albeit rarely.

The accuracy of the calculated distances of influence in the Juktan area is supported by the fact that four wells in Renbergsbyn village were influenced by the tunnelling. The drawdown in these wells was between 25 and 40 m at a distance of between 1,500 m and 2,000 m away from the tunnel. This influence is reported in detail by A. Carlsson and T. Olsson (1978a:382 and 1979d:266).

Numerical calculations of the influence of the tunnelling

For the purpose of studying the effects of tunnelling on the groundwater system, calculations were performed according to a two dimensional, finite element method in a limited section of the Juktan area (Axelsson and Olsson 1979). The section chosen is situated near Lake Renbergssjön, as seen from Fig. 51. Three large zones of crushed rock intersect the area while the intervening rock mass is fairly homogeneous. From water leakage in the tunnels and data from wells drilled in rock, the hydraulic conductivity of the zones and the rock mass were determined as about 10^{-4} m/s and $2 \cdot 10^{-7}$ m/s respectively. Both recharge and discharge areas are present in the relevant section.

The numerical model used in this calculation is a computer-based program for analysis of groundwater flow by means of the finite element method. The program, GEOFEM-G, is included in a greater program, CHALMFEM, which is available at the computer centre at the University of Gothenburg (Runesson *et al.* 1978).

This program solves a differential equation of the following type:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) = S \cdot \frac{\partial h}{\partial t} + W \qquad (52)$$

where T_x and T_y = transmissivity in x- and y-directions (m^2/s)
h = groundwater level (m)
S = storage coefficient (l)
t = time (s)
W = recharge or discharge of groundwater (m^3/s)

The model presumes two-dimensional groundwater flow and a transmissivity independent of the groundwater level. The area to be calculated is divided into a number of elements forming a element-mesh. The sides of the elements are determined by an equation of the second degree which is defined by two corner-nodes and one middle-node. Within each element a value of the transmissivity is stated and, if transient calculations are to be made a value of the storage coefficient. The present calculations, however, were carried out solely as stationary state calculations. Recharge or discharge of groundwater may be attributed to any element, along the side of an element, or in a corner-node. Boundary condictions may be stated at the corner-nodes.

The mesh-system used in the present study is composed

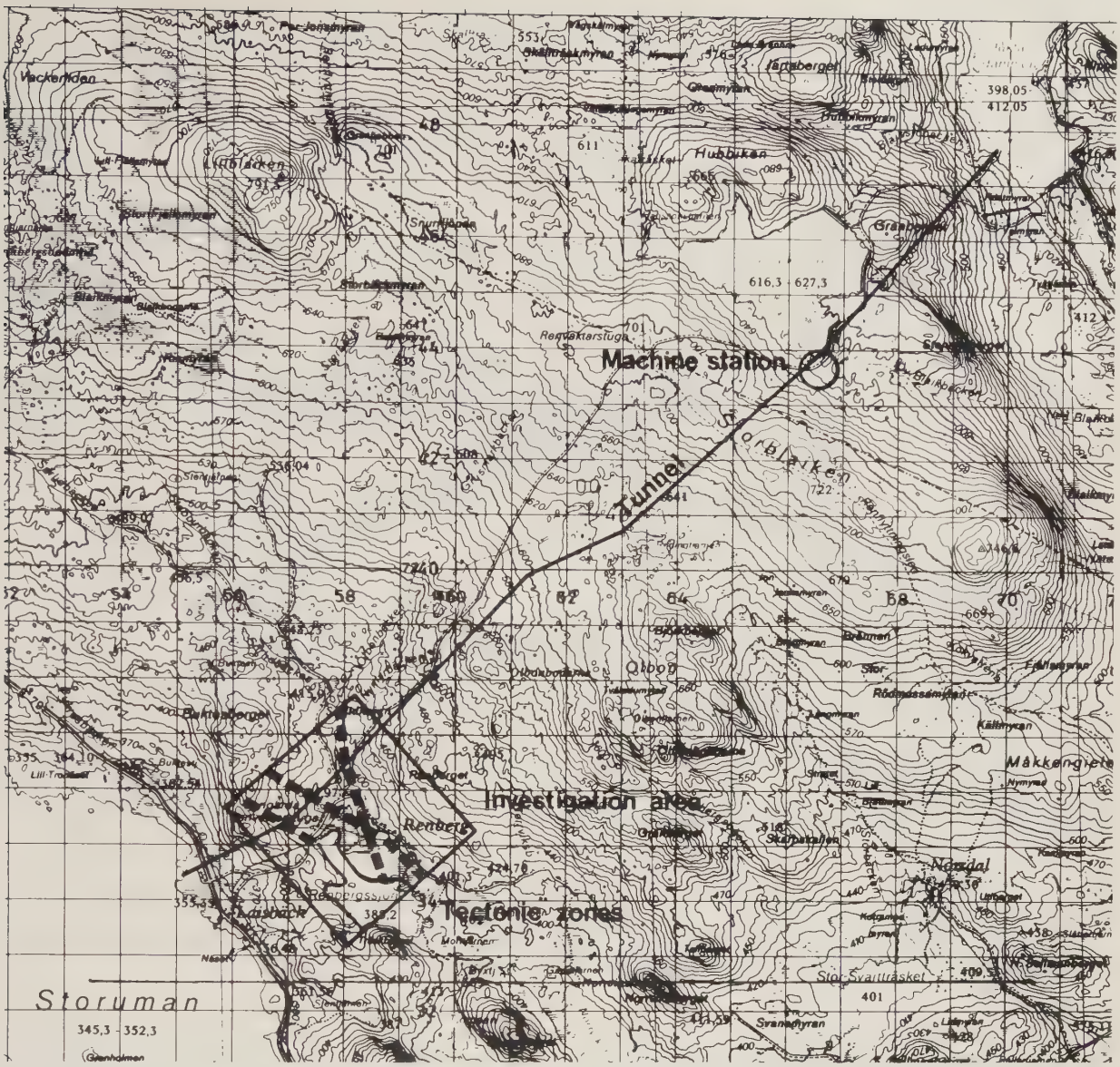


Fig. 51. Part of Topographical map 23 H Stensele showing the construction area of the Juktan power plant with the tunnel-system and the area at Renbergsbyn village which has been calculated by a numerical method. The scale of the map is given by the 1 km squares.

of 204 individual elements adapted to the local geological and hydrogeological conditions. Figure 52 shows the element mesh and the properties assigned to each element.

The calculations were made in two separate steps:

Natural undisturbed conditions

The tunnel cutting through the area

The first step was intended to determine the accuracy of the preliminary values of the conductivity and recharge, and to adjust these to obtain a groundwater situation similar to reality. The tunnelling and its effect on the groundwater system was simulated in the second step.

Table 17 shows the preliminary stated values of the conductivity and recharge, and the values obtained during the first step which gave a groundwater situation similar to reality. The obtained groundwater situation under natural, undisturbed conditions is shown in Fig. 53.

The calculations show that the groundwater recharge to the rock mass is of the same order of magnitude as the values of this parameter presented above. Indeed the recharge to the zones of crushed rock far exceeds the preliminary stated values. Presumably this is due partly to the fact that these zones have a higher direct infiltration capacity and partly to drainage of the surrounding rock mass. The obtained recharge values assumed to apply to those areas in which recharge is possible, but not the discharge areas. With an areal distribution of the discharge areas of 20 per cent, and considering the different recharges between the homogeneous rock mass and the tectonic zones, it is possible to determine an average recharge value valid for the entire calculated area. This mean value amounts to 7.3 mm/year, which is in very good agreement with the recharge obtained by other methods.

The conductivity values dervied from the numerical calculations differ from those preliminarily stated. This

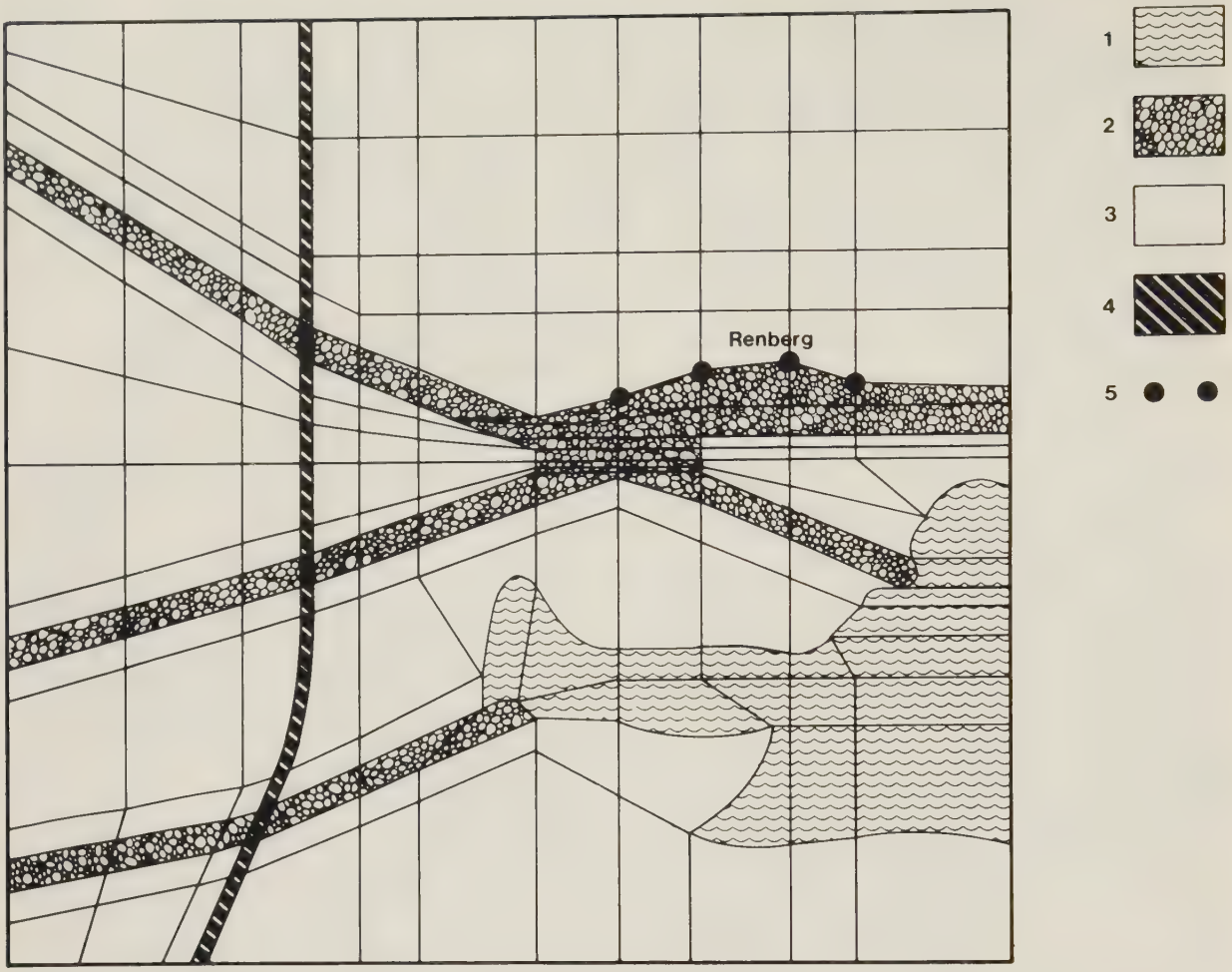


Fig. 52. The element mesh and the element properties in the calculated area. 1. Lake Renbergssjön. 2. Zones of crushed rock. 3. Rock mass. 4. Tunnel. 5. Wells drilled in rock. (C. Axelsson and T. Olsson 1979).

may be dependent on the possibilities of greater differentiation offered by the numerical calculation compared with the differentiation allowed by the analytical method.

The second step, in which the drainage due to the tunnel is simulated, is based on the values obtained in the first step. An additional control of the accuracy of the parameters is there given by the observed drawdown in the wells in Renbergsbyn village. The groundwater situation obtained in the second step is shown in Fig. 54. The groundwater level is considerably lowered along the tunnel and the three zones of crushed rock which intersect the area. Moreover the surrounding rock mass is drained by the conductive zones. Notwithstanding that two zones appear to be connected with Lake Renbergssjön, very little infiltration takes place. The majority of the groundwater originates outside the actual calculated area.

The recharge to the rock is increased by drainage by a factor of about 3 on average. This increase is greatest near the tunnel and the tectonic zones and decreases as the distance from the draining structures increases.

The results from the numerical calculations to a great extent verify those from the analytical calculations. Differences occur however, and the differentiation is far better with the numerical method. The reliability of the determinations of the hydraulic conductivity based on water pressure tests and leakage measurements also, seems fairly good.

Table 17. The hydraulic properties of the rock mass and zones of crushed rock in the area of Renbergsbyn Village. Preliminary in-put values and values adjusted as a result of the numerical calculations.

Parameter	In-put values	Adjusted values
Hydraulic conductivity		
Rock mass	$2 \cdot 10^{-7}$ m/s	$2 \cdot 10^{-8}$ m/s
Tectonic zones	$1 \cdot 10^{-4}$ m/s	10^{-5}–10^{-4} m/s
Groundwater recharge		
Rock mass	80 mm/year	5 mm/year
Tectonic zones	80 mm/year	50 mm/year

Groundwater balance under disturbed conditions

The natural groundwater discharge in the homogeneous rock mass amounts to about 5–10 mm/year which increases by a factor of three when affected by the drainage. The recharge into tectonic zones is calculated to be about

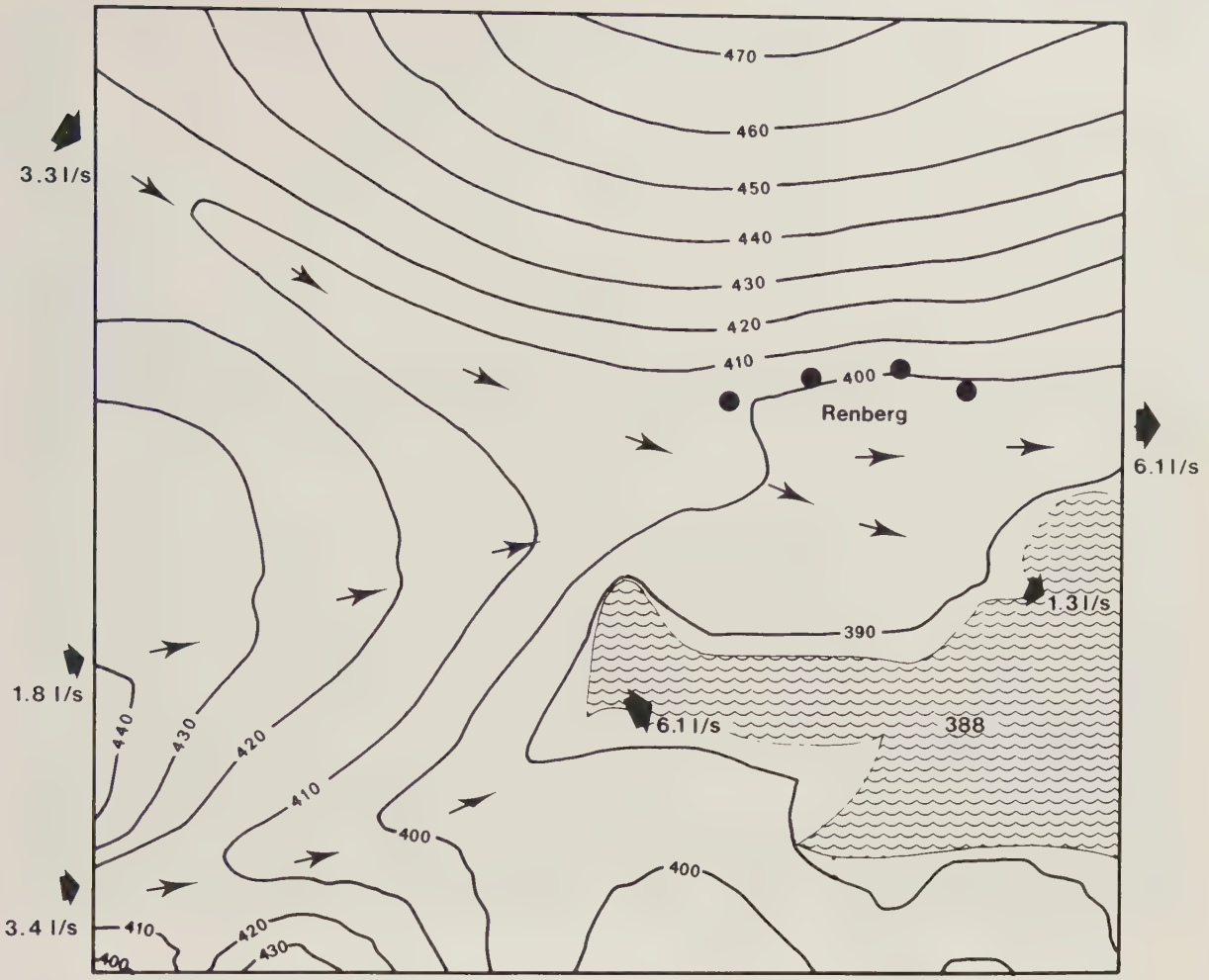


Fig. 53. Groundwater levels and flow directions under natural, undisturbed conditions (C. Axelsson and T. Olsson 1979).

50 mm/year under undisturbed conditions. This value also increases by a factor of about three when the rock is drained. With an area of about 50 km^2 influenced by the tunnelling, the total groundwater inflow into the tunnel-system would be about 70—100 l/s, which is much less than the reality (250 l/s).

The difference between the theoretical and the actual inflow is far too great to be attributed to a continuing drainage of the rock mass. By definition the largest leakages are generally stationary. This indicates the presence in the tunnel of water leakages which are in direct communication with surface water reservoirs.

This explanation is suggested in the description of the water inflow, and also indicated by the datings from the tunnel. One tarn in the tunnel-system has been lowered considerably as seen in Fig. 55, but this is the only direct evidence of influence on surface water reservoirs. The photograph was taken at a time when all other lakes and tarns in the area were at a maximum level.

The recharge into the rock must increase by a factor of about 3 if the theoretical inflow is to be in balance with the actual. In other words a recharge of about 45—90 mm/year in the homogeneous rock mass and about 450 mm/year in tectonic zones. These values, however, are much too high for accuracy. The rock cannot absorb such a large quantity of water (cf. C. Axelsson and L. Carlsson 1979:39). However, the quality and continuity of the zones outside the investigation area also may be of importance, which affect the distance of influence and consequently the total recharge within the influenced area.

The most plausible explanation therefore seems to be that the values presented for conductivity and recharge are fairly accurate, and the discrepancy depends on infiltration of surface water into the rock. The extra recharge is probably most important in the greater water-bearing zones.

Technical problems related to groundwater in connection with subterranean constructions

Construction works below the groundwater level usually cause inflows of groundwater. The quantity of the water inflows may vary considerably, from almost completely dry rock without noticeable inflow up to inrushes of several m^3/min.

The problems arising from the presence of groundwater may be divided into two groups as in Fig. 56. This figure illustrates adverse effects of the disturbance of the

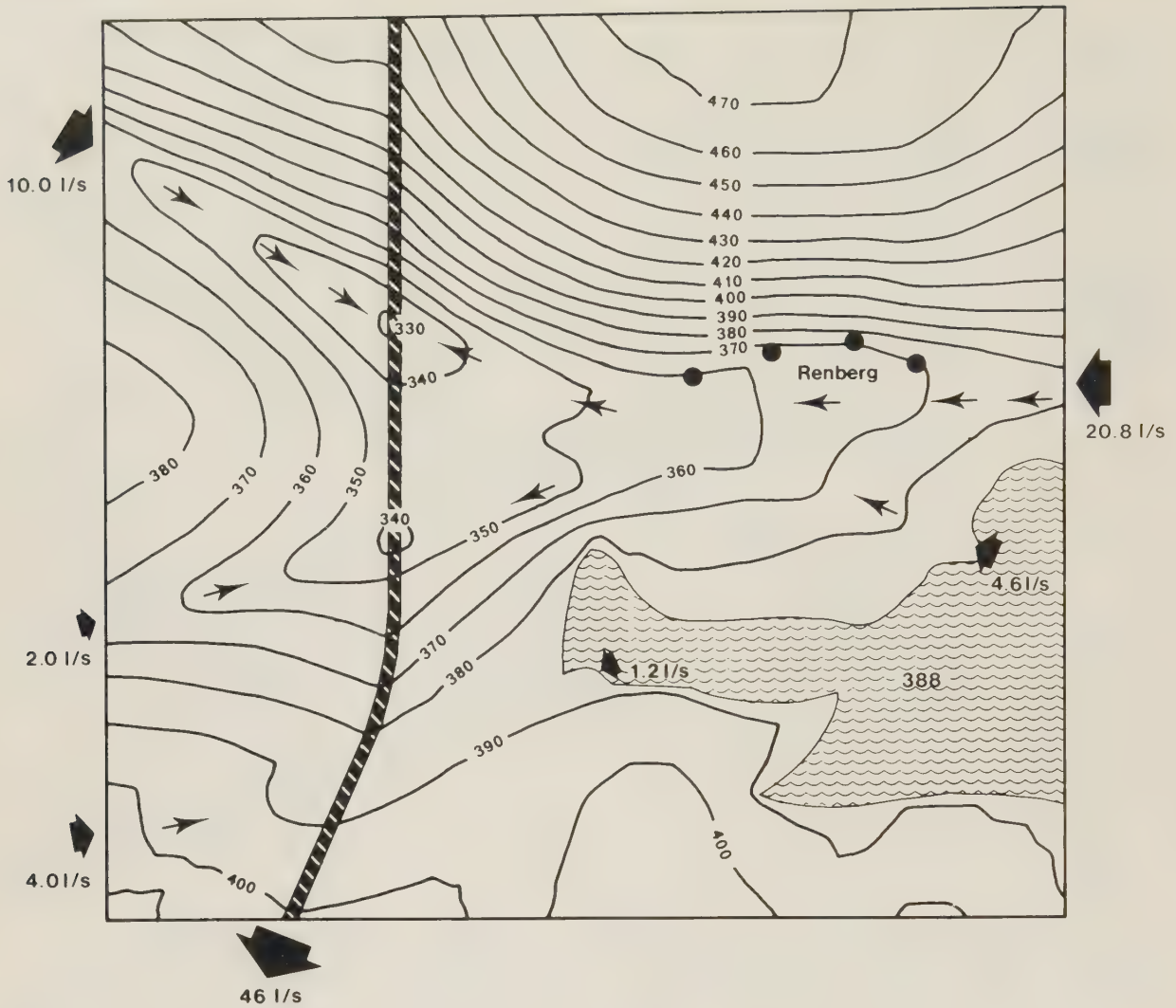


Fig. 54. Groundwater levels and flow directions when the area is disturbed by tunnelling (C. Axelsson and T. Olsson 1979).

groundwater system, some of which depend primarily on the actual water inflow while others are caused by the resulting groundwater drainage.

The extent of these problems depends on the quantity of leakage. Rather large quantities of water inflow may occur without serious interference with construction works, rock stability and drainage pumping. But even very small inflows may damage the instrumentation and machinery installed in the rock chambers, thus disturbing the function of the construction. Small water inflows may also cause groundwater lowering and land subsidence in the vicinity of the construction.

The main problem in connection with subterranean constructions in urban areas is the induced consolidation of clay soils due to the lowering of the groundwater. The relation between subterranean constructions and land subsidence have long been observed, but became generally accepted only in recent decades. Damage due to lowering of the groundwater level is reported by e.g. Morfeldt (1967), Bjurström (1977) and Broms *et al.* (1977).

The geographical location of the Juktan plant, however, produced few environmental problems despite the wide extent of the areas of influence, but serious problems arose in the tunnels, due partly to the water inflow and partly to very high hydraulic pressures in the rock mass.

Environmental effects

It is obvious that the groundwater system in which the tunnels are situated was seriously disturbed by the water inflow and the resulting drainage of the rock mass. As shown above, the groundwater level was considerably lowered at great distances from the tunnels.

But as the Juktan plant is situated in a rural area without compressive soils, no negative land subsidence ensued. The only noticeable effect proved to be a groundwater lowering in wells for water supply in the village of Renbergsbyn. Four wells sunk in sand were here emptied by the drainage, whereupon they were replaced by four wells drilled in rock. These new wells were also considerably lowered and the maximum recorded drawdown amounts to 38 m. This large drawdown is remarkable as

Fig. 55. A tarn, close to the intermediate access tunnel, in which the water level has been considerably lowered due to the dehydration of the tunnel. Photograph by A. Carlsson.

the wells are situated between 1,000 m and 2,000 m away from the tunnel. Nevertheless, this effect is in accordance with the calculated distances of influence along a zone of crushed rock near the wells. Figure 57 shows the wells in relation to the tailrace tunnel and the area of influence. Other than these water wells and a tarn at Mt Storblaiken, no noticeable effects on the environment were found. Yet the groundwater drainage probably caused other effects. Discharge areas penetrated by the areas of influence probably changed into recharge areas as the groundwater level in the rock mass was lowered. Moreover a gradual change in microclimate and in vegetation may ensue. Such effects, however, are not known in the Juktan area.

Effects on the construction

The heavy groundwater inflow into the tunnel system caused many problems with the construction work and the rock stability.

The data on the progress of excavation until December 1977 are summarized in Table 18. The excavation was severely disturbed by areas with poor rock quality together with water inflow. A great number of cast-in-place concrete arches were here constructed, but other tunnel supports such as rock-bolting affected the excavation too.

The most extensive tunnel supports were inserted in the huge zones of crushed rock in the tailrace tunnel (marked A and B in Fig. 6). But the reinforcements in these zones were prompted mainly by the rock quality and only to a small extent by the groundwater. The groundwater may, however, have been of importance in that the water pressure caused a reduction of the shear strength of the fractures. Figure 58 shows some arches in these heavily supported zones.

In zone D in the tailrace tunnel the groundwater was a primary cause of the difficulties. In this zone clay filled fractures occur and the high water pressure (initially about 300 m) broke through a clay filled fracture, causing a considerable inrush of water. An attempt was made to reduce the water inflow by grouting, but the hydraulic head increased as a result, and a renewed break-through ensued. The grouting was then stopped.

In the cable shaft in the machine hall area where the leakage E occurred, extensive grouting was undertaken to reduce the inflow capacity. The reason for this grouting was that the water could inflict damage on the cables in the shaft. The inflow was reduced by grouting from an initial 20 l/s to about 2 l/s. Figure 59 shows the different phases of the grouting and the resulting reduction in inflow capacity.

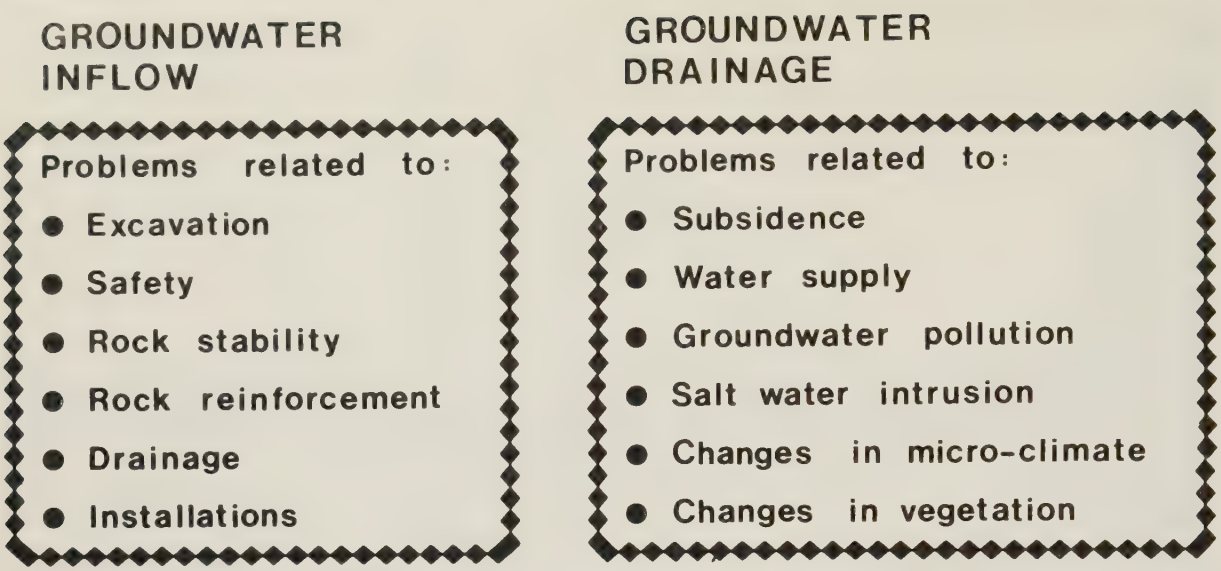


Fig. 56. The effects of groundwater leakage. Some of the problems are related to the groundwater inflow while others are caused by the resulting groundwater drainage.

This brief review of the excavation illustrates that water combined with poor rock quality caused serious problems in the Juktan tunnels. It is also seen from Table 18 that the tunnels with the highest water inflows, i.e. the access and the tailrace tunnels, have the lowest actual progress. Poor rock quality per se can create difficulties, but the negative effects are aggravated if water is present. The main reason is that the water pressure in the fractures reduces the normal stress in the fractures and the shear strength is thereby greatly diminished. This may be summarized as follows: the higher the pressure the greater the problem.

The location of the plant, deep in the rock mass, is the prime factor as regards the high hydrostatic pressure which prevailed in the rock mass above the excavations. In the machine hall area the pressure was constant at about 2 MPa for a long time. As this high pressure could endanger the stability of the rock chambers attempts were made to reduce it. A great number of drill-holes were made in the roofs of the excavations in order to drain the neighbouring rock mass. These drill-holes improved the stability of the surrounding rock walls, but did not reduce the total hydraulic head. To obtain better stability, a drainage tunnel was designed, sited above the main excavations. The construction of this tunnel, coincided with the drilling for one of the surge shafts. As this drill-hole penetrated a small water-bearing zone with an inflow of about 4–5 l/s, the hydraulic head was suddenly reduced to about 0.6 MPa. The pressure release continued to a stationary level of about 0.2 MPa. The construction of the drainage tunnel was then stopped.

Conclusions and final remarks

The results are of a quantitative interest due to the exceptionally heavy water inflows and high hydrostatic pressures present in the area. The investigations justified the statement of some fundamental principles of rock hydraulics. The results showed the way in which different hydraulic processes cooperate in a given hydraulic situation.

Due to the possibility of checking hypotheses by means of measurements and hydrogeological calculations, the values of the different parameters of importance for the water-bearing ability of the formations could be determined with a high degree of accuracy.

The investigations proved the vital point that the recharge due to precipitation infiltration is the parameter most important for the groundwater system. The determination of the recharge to the till aquifer is beset by uncertainty, because of its dependence on both hydro-

Table 18. Average monthly excavation of the Juktan tunnels and tunnel supports of cast-in-place concrete arches per December 1977 (A. Carlsson and T. Olsson 1978a).

Tunnels	Length driven December 1977	Prognosticated average excavation progress	Average excavation progress Result		Zones with arches of cast-in-place concrete	
	m	m/month	m/month	%	m	%
Headrace	4930	150	142	95	5	0.1
Access tunnel	1780	125	96	77	12	0.7
Intermediate	1325	125	125	100	0	0
Tailrace	12995	135	98	73	350	2.7

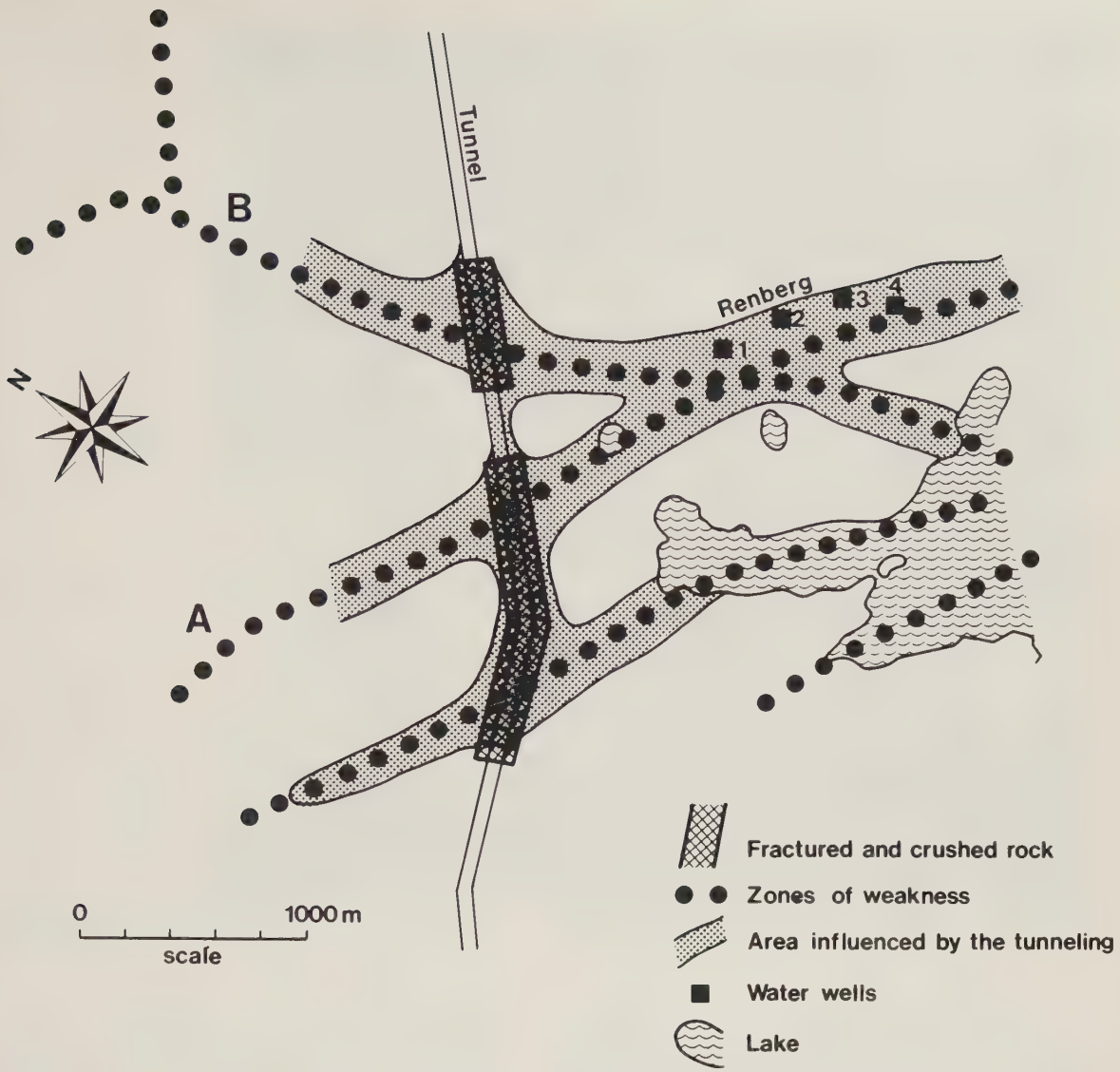


Fig. 57. The area at Renbergsbyn Village with the tailrace tunnel, zones of crushed rock and water wells.

geological and hydrometeorological conditions. The hydrogeological conditions govern the infiltration capacity while the hydrometeorological conditions determine the amount of water available for infiltration.

On the other hand, the recharge to the rock aquifer is in general more narrowly defined, being governed mainly by the hydrogeological conditions. The volume of water available in the soil cover is normally greater than the amount of which may infiltrate. This holds good when the rock mass is covered by soils, in which case the recharge to the rock may be at its maximum. In undisturbed conditions the soil and the rock aquifers in the long-term are on balance. Groundwater extraction from the rock will result in an increased recharge, while the soil aquifer will act as an infiltration reservoir. The increase in recharge takes place both in the homogeneous rock mass and in zones of fractured and crushed rock.

In exposed bedrock the recharge into the homogeneous rock mass will probably be of a very small magnitude and the precipitation results in a surface run off. The homogeneous rock mass will probably be fed to a high degree by groundwater from the tectonic zones. In diagram form it is then possible to characterize zones of fractured and crushed rock in the same manner as eskers, i.e. in leaking and draining zones.

No accurate methods exist of determining the groundwater recharge. The method used in this paper to determine the recharge by analysing the aquifer response due to a specific precipitation occasion, may perhaps be applied to obtain reliable results. Yet fairly wide knowledge of the soil water conditions is required, as the water stored in the soil acts as a buffer, which results in a delayed and leveled response. Furthermore, the calculations must be based on periods of days or shorter to obtain the best result.

The investigations have shown that the main part of the available precipitation participates in the recharge process. However, the available precipitation occurs to a great extent as groundwater limited by place and by time, after which it discharges and runs off as surface water (cf. Holmbom and Larsson 1977).

Laboratory determinations of the hydraulic conductivi-

Fig. 58. Tunnel supports of cast-in-place concrete arches in the tailrace tunnel. Note the groundwater inflow at the tunnel face in the central of the photograph. Photograph by Ö. Eriksson.

ty of till material give unduly low values compared with the natural, undisturbed till. The reason for this discrepancy seems to be the stratification in the natural till deposit. The structural discontinuities in the till, however, are probably not interconnected, which means that the higher conductivity is only of local importance. On a regional scale the till material with its low permeability will govern the groundwater flow. On a local scale the discontinuities seem to be of importance for the fast local groundwater flow induced by high recharge. The till formations should therefore be regarded as composed of a great number of isolated aquifers rather than as a single interconnected water-bearing unit.

The conductivity of the rock mass varies widely within the area. Blocks of rock poor in fractures, and consequently with low conductivity, alternate with zones of fractured and crushed rock with high conductivity. The variations obtained are much greater in the rock aquifer than in the till deposits, primarily because of differences in conductivity between the rock and till matrices. The conductivity values of the rock mass cover over six decades, and this range will become even greater if the differentiation between the rock matrix and the discontinuities increases. The difference in conductivity between the rock matrix and the single discontinuities is about 10 to 12 decades. In the till deposits the corresponding range is only about two to four decades.

The great difference in conductivity between the homogeneous rock mass and tectonic zones implies that the bedrock is acting hydraulically as a multiple aquifer system (A. Carlsson and Olsson 1979b). Different water-bearing units exist side by side without noticeable interaction. Disturbances in one unit will only influence that unit and the surrounding rock mass. Thus the more intact surrounding rock constitutes impermeable boundaries or layers between the water-bearing zones.

Due to the high conductivity in zones of fractured and crushed rock, the pressure differences will rapidly come into balance. This effect of the tectonic zones seems to be of prime significance for the water balance in the area.

The humid climate in Sweden, with usually a greater volume of water available for infiltration than can be absorbed by the rock, results in a groundwater level close to the ground surface.

The hydraulic conductivity of the rock mass decreases with increasing depth below the rock surface, probably because of the concomitant change in the rock stress conditions. The depth-dependence seems to be more distinct in the homogeneous rock mass than in zones of fractured and crushed rock. However, the data base is too small to verify this condition.

The hydraulic anisotropy prevailing in Revsund granite is probably due to the total stress field. The anisotropy seems to be induced by differentces between the primary

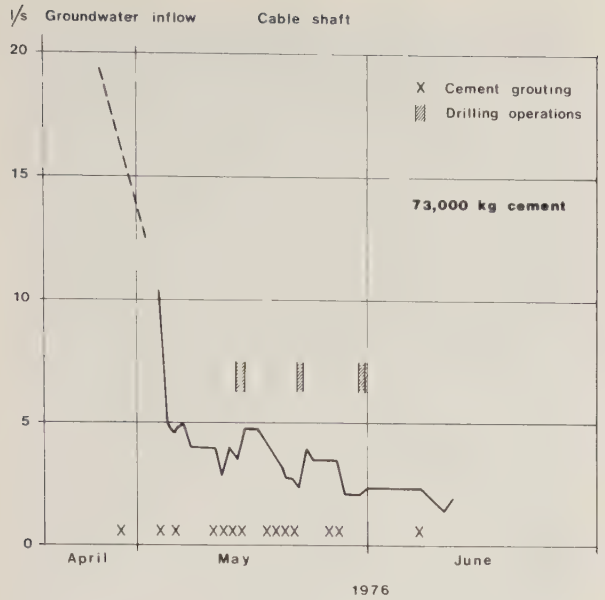


Fig. 59. Grouting of the water inflow in the cable shaft and its effect on the groundwater inflow.

stresses in the rock with a higher conductivity perpendicular to the direction of the smallest principal stress and parallel to the direction of the greatest stress. It seems reasonable to assume that the stress-induced anisotropy will be most distinct in unfoliated rock types. The hydraulic anisotropy in rock masses with a well developed foliation is probably only to a minor extent dependent on the stress distribution, being more subjected to the structural anisotropy. This indicates that the rock stress measurement may be an useful tool for determination of the hydraulic anisotropy in massive rock types such as granite.

The studies of the groundwater flow point to that the water-bearing tectonic zones are vital for the flow, not only because of their ability to conduct large amounts of water but also by reason of their function as controlling units for the groundwater levels and flow pattern in the area. Moreover, the topography also plays part in the hydrogeological conditions.

A generally accepted condition as regards the groundwater recharge in different topographical situations, is that the recharge is lower in high altitude areas. This is attributed to the fact that the conditions for surface run off are favourable, due to the steep inclination of the ground surface. Consequently, the recharge is presumed great in flat level areas. These conditions are correct under certain circumstances. The hydrogeological and hydrometeorological conditions must in this case be so correlated as to permit the aquifer to receive a larger quantity of water than the available precipitation. However this type of climate and this type of aquifer do not exist in Sweden, with its crystalline basement. In Sweden, the recharge to the rock is governed mainly by hydraulic factors, which results in a greater recharge in hilly terrain than on flat land. The reason is that steeper hydraulic gradients prevail in hilly terrain, which give rise to a heavier groundwater flow compared with the flow obtained on the flat land with its gentler gradients. Consequently, according to the law of continuity, the recharge must balance the discharge, which gives a higher recharge in the hilly terrain. This is valid under undisturbed conditions. Disturbances due to water extraction will mean that a greater induced recharge will be obtained in flat terrain.

In consequence of the decrease in the hydraulic conductivity and the estimated hydraulic gradient with increasing depth the groundwater flow too decreases with increasing depth. The results show that about 59 per cent of the total groundwater flow in the rock takes place within the uppermost 10 m of the rock mass, only about 2 per cent occurring below the 200 m-level. This circumstance greatly affects the transformation velocity. While the transformation is rapid in the uppermost part of the rock—the transfer of the groundwater in the top 10 m takes a few years—the time needed for transformation at deeper depths increase considerably with incresing depth. This was verified by the radiocarbon datings of the groundwater from the tunnels.

The tunnelling and the groundwater inflow demonstrated that the groundwater system may be influenced at great distances from the tunnel. Long distances of influence are found, especially along tectonic zones, and the groundwater within the homogeneous rock mass is considerably affected by the tunnelling. Due to the groundwater lowering an increase in recharge is obtained. The reason for this induced recharge is probably the change of the pressure conditions in the rock mass which lead to an increased pressure difference between the rock mass and the soil cover.

The final conclusion from the present study is that data obtained in a hydrogeological investigation must be handled with care. The scale in which the determinations are made exerts a great influence on the validity of the data, and results obtained in one scale will not necessarily be valid in another. The differences in representativity are great between results from laboratory tests or drillings versus those from tests made in till pits or rock chambers.

Acknowledgements. Professor Lars-König Königsson, Head of the Department of Quaternary Geology, University of Uppsala, made this work a reality. His great support has encouraged me to complete this thesis.

My co-operation through the years with my dear friend and colleague Anders Carlsson, Swedish State Power Board, proved invaluable. Mr Cårlsson has had a most important share in the field work and in the preparation of the present paper.

Without the neverfailing encouragement and enthusiasm of Dr Jüri Martna, Chief Engineering Geologist, and Mr Karl Arthur Scherman, Chief Engineer, Swedish State Power Board, Department of Civil Engineering, I could not have brought this work to completion. I offer them my sincere appreciation.

I am very grateful to Mr Olle Gellerstedt, Project Manager at Juktan and his colleagues Messrs Stig Fredriksson and Tage Rickardsson for the opportunity to carry out my work within the construction area. I am also grateful for the resources which they put at my disposal, namely necessary equipment and a large number of measurements compiled by the staff at the construction site.

Dr Leif Carlsson, Geological Survey of Sweden, Dr Rune Gustavsson, Department of Mathematics, University of Uppsala, Dr Jüri Martna, State Power Board, and Dr Kennert Röshoff, University of Luleå, have read and criticized the manuscript and also offered valuable comments on the results and the theories. Thanks are also due to Mr Martin Moberg, State Power Board, who put at my disposal the bulk of the documentation from the site exploration. I discussed several problems with him and he gave me good advice on many occasions.

The illustrations were re-drawn by Miss Kerstin Orvandt and Mrs Anne-Christine Sjöberg, Geological Survey, who did an excellent job despite the great difficulties in the translation of my hieroglyphics into notations in Arabic letters. The manuscript

was linguistically corrected by Mrs T.J.M. Gray, B.A., T.K., Uppsala.

To all these persons and to Dr Harald Agrell, Mr Charlie Axelsson, Dr Jan De Geer, Mr Torbjörn Fagerlind, Mr Lennart Karlqvist, Professor Gert Knutsson, Dr Gösta Persson, Mr Åke Redin and Mr Oscar Rinne and to all other colleagues I tender my sincere thanks.

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ISBN 91-7388-015-9
ISSN 0345-0074
Textgruppen i Uppsala AB